# REPORT

## Northern Interceptor Coastal Processes Report

**Prepared for:**

**Watercare Services Ltd**

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ENVIRONMENTAL AND ENGINEERING CONSULTANTS

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# Executive summary

Tonkin and Taylor Ltd (T&T) has been commissioned by Watercare Services Limited (Watercare) to assess the potential coastal process effects related to the construction, operation and maintenance of the proposed Northern Interceptor. The key elements relevant to this coastal process study include:

- Installation of pipelines in a widened section of the existing motorway causeway
- Installation of dual pipelines across the Upper Waitemata Harbour to Greenhithe via marine trenching or horizontal directional drilling ("HDD")
- Installation of dual pipelines under Te Wharau Creek via HDD.

## Physical setting and key processes

The Upper Waitemata Harbour area bounded by Hobsonville Point, Greenhithe and Herald Island is a relatively low energy environment dominated by tidal flow concentrations through the narrow channel and wind generated wave conditions on the intertidal sand flats.

Historic studies have indicated long term sedimentation patterns in sheltered areas, particularly within the numerous inlets and creeks (Hume, 1983) although in the more exposed areas, subject to higher wave and tidal flows, there is no clear trend. The results of our review of the available photographs, particularly since the duplication of the bridge was completed, show less than minor changes in coastal processes with changes mainly associated with the increase in mangrove growth.

Currents within the main body of the Upper Waitemata Harbour are driven principally by the tide and current patterns are governed mainly by local bathymetry. Hydrodynamic modelling shows highest currents are in the main channel adjacent to the Greenhithe shoreline (greater than 0.8 m/s) and lower currents are present on the intertidal areas. There are slightly higher ebb velocities (up to 0.3 m/s) than occur during flood tides (up to 0.2 m/s) due to the sheltering nature of the causeway during flood conditions compared to ebb conditions.

Maximum significant wave heights of between 0.5 m and 0.71 m are possible at this location, with periods of between 2 and 3 seconds along the causeway. During south easterly conditions wave heights could reach up to 1.0 m within the channel with periods of around 3.4 seconds.

## Construction methods

The pipes crossing the harbour will either be installed by horizontal directional drilling (HDD) or by marine trenching. Due to the differing requirements of both methods, there are different alignments proposed for these options.

For HDD all works will be carried out on established land areas on the widened causeway and at Rahui Road and the route will be direct. The drilling will locate the pipeline in competent ground below the seabed level. Key risks of this methodology is accidental and uncontrolled spillage of drill water into the CMA. It is assumed that methods to prevent this likelihood will be included in a Construction Management Plan (CMP), prepared by the selected tenderer.

For the marine trenching method the alignment is predominantly along the main channel alignment, with connections to land across the intertidal area at Rahui Road. Due to the shallow nature of the harbour, separate construction methodologies have been developed for the marine trenching option for the intertidal areas and the installation of the pipeline across the main channel, with specific consideration of the interfacing between these two areas to connect the pipe.

In the intertidal areas the trenching and pipe laying will be done by land based equipment from a temporary access bund. The excavated material will be disposed offsite and the trench backfilled with clean granular fill and topped with rock armour.

In the main channel alternative methods such as a jet trenching (fluidisation), or a mass flow excavation using low head fan unit could be used. These devices are towed by a barge will be used to either sink the pipe through the marine sediments to the required depth or to form a trench in which the pipeline can settle. In the case of the mass excavation method, backfilling of the formed trench may be required.

## Assessment of effects

There should be no long-term effect of the proposed works on the physical coastal environment with either proposed method of installation.

If the pipes are installed by HDD then there are no short-term effects on coastal processes as the pipe is situated sufficiently below the level of the seabed.

The pipe trenching option comprising a combination of open trenching along the intertidal area and hydraulic jetting or mass excavation within the channel will have potential short term effects. Short term effects on the intertidal area include localised increases in suspended sediment concentrations along the edge of the temporary stockpile of excavated sand. This is likely to be of a similar order of magnitude as the natural rates of suspended sediment along the intertidal area when wind generated waves exceed 0.2 m in height.

In the subtidal area the hydraulic jetting/mass evacuation will result in significant rates of suspended sediment concentration over a short period of time (less than 1 to 2 tidal cycles) at the location of the jetting head. The mass evacuation option is likely to result in a greater spatial extent of elevated suspended sediment concentrations compared to the hydraulic jetting with the potential for greater short term deposition on the adjacent intertidal area, but still only occurring over several tidal cycles.

## Mitigation

The main option to mitigate potential short term effects on high rates of suspended sediment concentrations would be to complete the pipe installation through HDD techniques under the harbour.

However, the proposed trenching methods for the intertidal and the interface works to connect the intertidal and subtidal works are relatively small scale and localised. If required, the extent of disturbance could be confined with the installation of a silt fence. The silt fence could be fixed on the seabed for the intertidal work, but would need to be sufficiently robust to withstand localised wind generated waves. For the interfacing connection, works would be carried out at high tides when tidal currents were low to avoid the requirements of a floating silt curtain.

For the subtidal hydraulic jetting/mass flow excavation will create greater levels of suspended sediment migration. However, these high sediment loads are of a short duration and are localised to the vicinity of the jetting head and its surrounds.

A mobile floating silt curtain is not a viable method to reduce the potential visible silt plume for the upper part of the water column. A fixed floating silt curtain around the entire perimeter of the works is also unlikely to be viable as it would create significant obstruction to other users in the harbour and is not considered practicable.

The most effective method for reducing the extent of the suspended sediment concentration could be reduced by limiting work to periods of slack high tide (i.e. 2 hours either side of high or low water if the size of plant allows) where tidal currents are low.  This would require a longer construction period and a greater duration of disturbance.

# 1 Introduction

Watercare Services Limited ("Watercare") is proposing to build new wastewater pipelines and associated infrastructure to convey wastewater from north-western parts of Auckland to the Rosedale Wastewater Treatment Plant ("WWTP") in Albany. This project is known as the "Northern Interceptor". Construction of the Northern Interceptor is intended to be staged, with the timing of various stages depending on the rate of population growth.

Tonkin & Taylor Ltd (T&T) has been commissioned by Watercare to assess the potential physical coastal processes effects related to the construction, operation and maintenance of the proposed Northern Interceptor Phase 1 ("the Project").

The proposed work requires various resource consents under the Resource Management Act 1991 ("RMA"). This technical report provides specialist input for the Northern Interceptor Phase 1 – Assessment of Effects on the Environment report ("the main AEE") prepared by MWH New Zealand Limited, which supports the resource consent application.

This report provides the following:

- A brief overview of the proposed works (in Section 2);
- A description of the environmental baseline for the physical coastal environment potentially affected by the project (in Section 3);
- Description of specific aspects of the project in relation to the physical coastal processes (in Section 4);
- Description of the investigations undertaken to assess potential effects (Section 5 and Appendices B, C and D);
- An assessment of the actual or potential effects on the environment (construction, operation and maintenance), having reference to the statutory framework and any other environmental factors considered relevant. This includes the identification of activities that could result in adverse effects and, in turn, identifying design refinements or construction methodologies that could avoid, remedy or mitigate such effects (Section 5);
- Recommended mitigation and management measures (Section 6).

# 2 Proposed works

The proposed Northern Interceptor Phase 1 will transfer existing flows from the Hobsonville Pump Station to the Rosedale WWTP. The proposed route is from the existing Hobsonville Pump Station, under the State Highway 18 motorway, along the northern side of the motorway causeway, and then under the Upper Waitemata Harbour, through Greenhithe and then the commercial area of Rosedale.

Key elements of the project include:

- Upgrading of the existing Hobsonville Pump Station
- Installation of a pipe under State Highway 18
- Installation of pipelines in a widened section of the existing motorway causeway
- Installation of dual pipelines across the Upper Waitemata Harbour to Greenhithe via marine trenching or horizontal directional drilling (“HDD”)
- Installation of dual pipelines under Te Wharau Creek via HDD
- Construction of a pipe bridge between Witton Place and North Shore Golf Course
- Installation of dual pipelines under Alexandra Stream via HDD
- Trenched construction for pipeline installation in roads, open space and other land
- Installation of associated infrastructure.

With the exception noted below, the proposed works are described in detail in the main AEE. The works described in the main AEE and shown on the drawings are assessed in this report.

Watercare is proposing some widening along the existing State Highway 18 motorway causeway near Hobsonville to provide for proposed water and wastewater infrastructure, including a section of the Northern Interceptor Phase 1 pipeline. That work forms part of Watercare’s proposed Greenhithe Bridge Watermain Duplication and Causeway project. That project is part of a separate resource consent package, and is described in a report titled Greenhithe Bridge Watermain Duplication and Causeway – Assessment of Effects on the Environment, prepared by Aecom New Zealand.

# 3 Environmental baseline

## 3.1 General setting

The Upper Waitemata Harbour is at the head of a complex, deeply indented and infilled drowned-river-valley estuary. Seven shallow tidal creeks (Hellyers, Lucas, Paremoremo, Rangitopuni, Brighams, Rarawaru, Waiarohia) drain into the main body of the Upper Waitemata Harbour, which in turn connects with the relatively broad and open Middle Waitemata Harbour (refer Figure 3-1). The basement rock of the Upper Waitemata Harbour, over which modern sediments have been deposited, is uneven and irregular. Sediments in the upper reaches of the 7 tidal creeks tend to be thinly draped, but in the lower reaches of the creeks, mudbanks exceed 10 m thickness in parts, and tend to be stabilised by mangroves along the landward margins (NIWA, 2004).

The creeks are largely intertidal, with narrow central channels. Approximately 50% of the Upper Waitemata Harbour is intertidal, and the seven tidal creeks account for approximately 50% of the area of the Upper Waitemata Harbour. At high tide, water depth over the intertidal areas is 1–2 m. Tidal currents are typically not strong enough over the intertidal flats to entrain bed sediments. The estuary sits within a valley that is aligned east–west, which crosses the dominant wind directions (southwest and northeast). Hence, it is relatively sheltered, and waves are typically small (NIWA, 2004).


*Figure 3-1: Site location*

## 3.2 Site description

The coastal environment includes the upper extent of the Waitemata Harbour and Lucas Creek. The main work area extends from the Upper Harbour Motorway to the reserve at the end of Rahui Road, Greenhithe. The Upper Harbour Corridor (SH18) is the main east/west link between Waitakere and North Shore running from SH16 in the west to SH1 in the east. The 457 m long Upper Harbour Bridge crosses the Upper Waitemata Harbour at its narrowest section. The original bridge and causeway reclamation was completed in 1974 and the duplicate bridge and widening of the causeway was completed in 2006. The site location is shown in **Error! Reference ource not found.**.

## 3.3 Bathymetry

The bathymetry of the Upper Harbour environs as well as Lucas and Te Wharau Creek is provided in hydrographic charts (refer Figure 3-2). This chart is made up of surveys carried out in 1958-1959 (Upper Waitemata Harbour) and from 1967 to 1987 (Middle Waitemata Harbour). More detailed hydrographic surveys of the main harbour pipe crossing was carried out as part of the present study and is reported in the Hydrodynamic Model Study (Appendix B). The additional survey information in the vicinity of the bridge and causeway was carried out by Scantec Ltd. This survey data is presented in terms of AVD-46 datum, some 1.743 m higher than Chart Datum. The more detailed survey shows very similar general trends to the hydrographic chart in this area suggesting no significant change occurred from the 1960's to the present at this location.

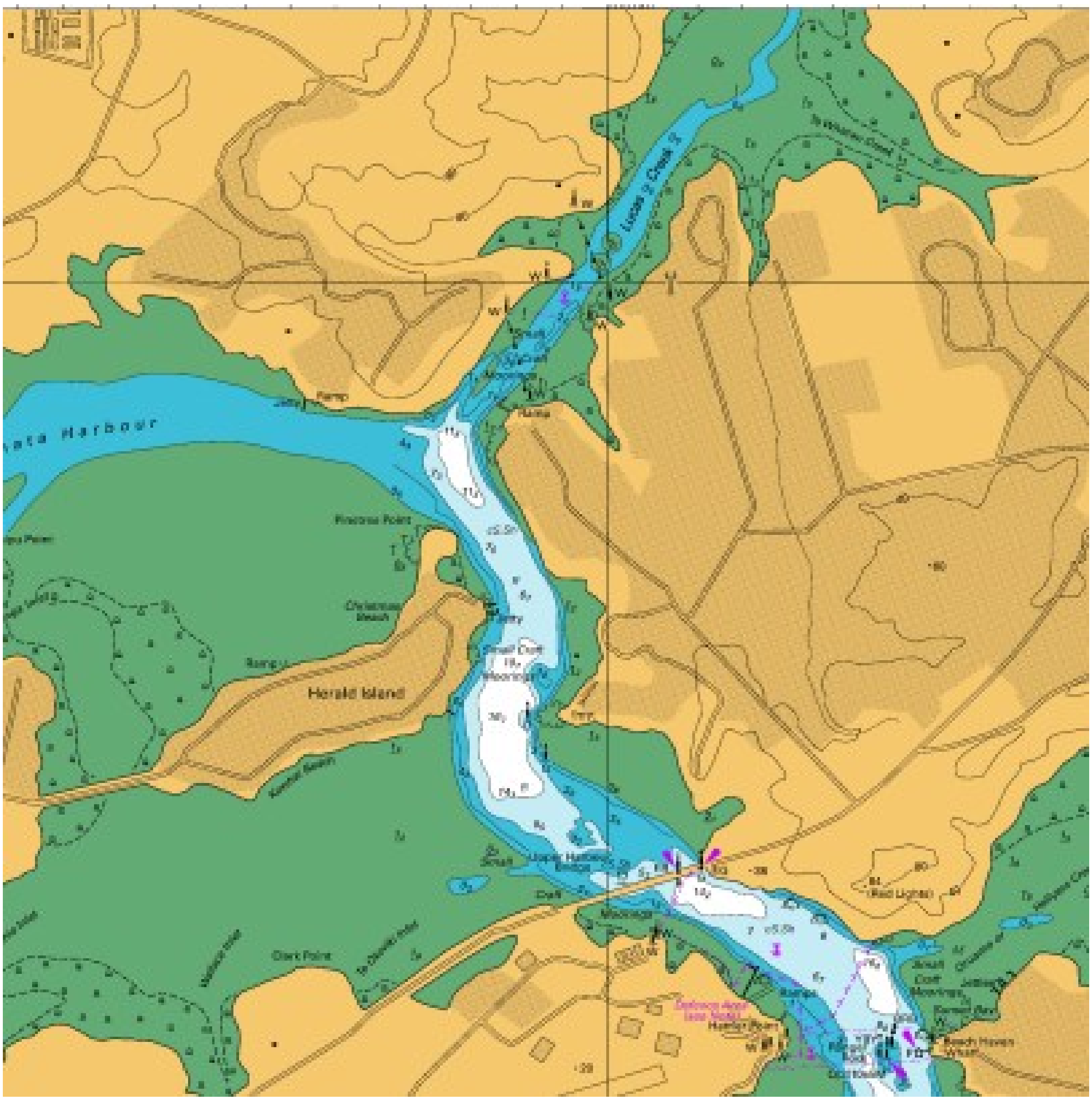



*Figure 3-2: Extract from hydrographic chart (NZ 5323), depths to Chart Datum*

Both the hydrographic chart and the Scantec Ltd surveys show a narrow channel with the narrowest point between the Hobsonville and Greenhithe headlands being around 250 m wide and the main channel some 200 m wide. The main navigation channel is close to the northern shoreline. There is a relatively deep ebb flow scour hole to the east of the bridge with a maximum depth of around 14.2 m below Chart Datum, although depths below the bridge are typically between 5 and 10 m below Chart Datum. The intertidal sand flats to the west of the causeway are around 0.3 to 1.8 m above Chart Datum.

An unusual feature of the Upper Waitemata Harbour is the rock sills within the intertidal zone of the Rangitopuni and Lucas Creeks. During much of the tidal cycle, these rock sills prevent upstream intrusion of estuarine water (NIWA, 2004).

## 3.4 Sedimentation trends

Sediments entering the Upper Waitemata Harbour are mainly sourced from soil and rock erosion within the Upper Waitemata Harbour catchment and carried to the estuary by streamflow, mainly during floods. Additional sources are erosion of stream channels and suspended-sediment entering from the Middle Waitemata Harbour on flood tides, but both of these are relatively insignificant (NIWA, 2004).

Sediment thickness throughout the harbour can be measured against the depth to the underlying Pleistocene basement rock. Depths can vary from zero at stream headwaters to over 10 m in relict river channels formed when sea levels were more than 120 m below present levels (6000 to 16000 years before present). Marine source sedimentation started some 6000 years before present (Hume, 1983). Much of this sediment can probably be attributed to the impact of man from about 1070 years BP (before present), with a major increase in the rate of sedimentation that coincided with the start of European settlement 100 years BP (Auckland Regional Water Board, 1983).

Core samples taken by NIWA Ecosystems (1997) show that sedimentation in the Upper Waitemata Harbour before humans colonised New Zealand was around 0.03-0.04mm/year. This increased with the arrival of Polynesian settlers due to clearing of forests, and increased by 2-3 times upon the arrival of Europeans, who logged Kauri forests and carried out gum digging (NIWA Ecosystems, 1997). Large scale land use changes associated with farming and urban development after 1910 further increased sedimentation, to produce current average rates of 2.0 mm/year in Lucas Creek and 1.4 mm/year in Brigham's Creek. The mud flats of these creeks are more prone to sediment accumulation, and show much higher rates (6-9 mm/year) from 1950 onwards.

Pollen dating and radioisotope dating by Swales et al. (2002) found that approximately 300 mm of material has been deposited in the sub-tidal regions of the Waitemata Harbour since the early 1900's, and that the average rate of sediment accumulation since the 1950's has been approximately 3 mm per year.

An examination of net sedimentation in the Upper Waitemata Harbour was done by comparing 1854 (refer Figure C-1, Appendix C) and 1958-59 soundings (refer Figure 3-3). Errors are likely to be in the order of 0.7 m for the 1854 survey and 0.3 m for the 1979 data (Hume, 1983). The results show little change along the main channels and Lucas Creek and no change in the vicinity of the Upper Harbour Bridge. However, some of the large erosion values are likely to be attributed to channel migration rather than significant localised erosion. There are generally consistent trends of sedimentation in the majority of the tributaries to the harbour.

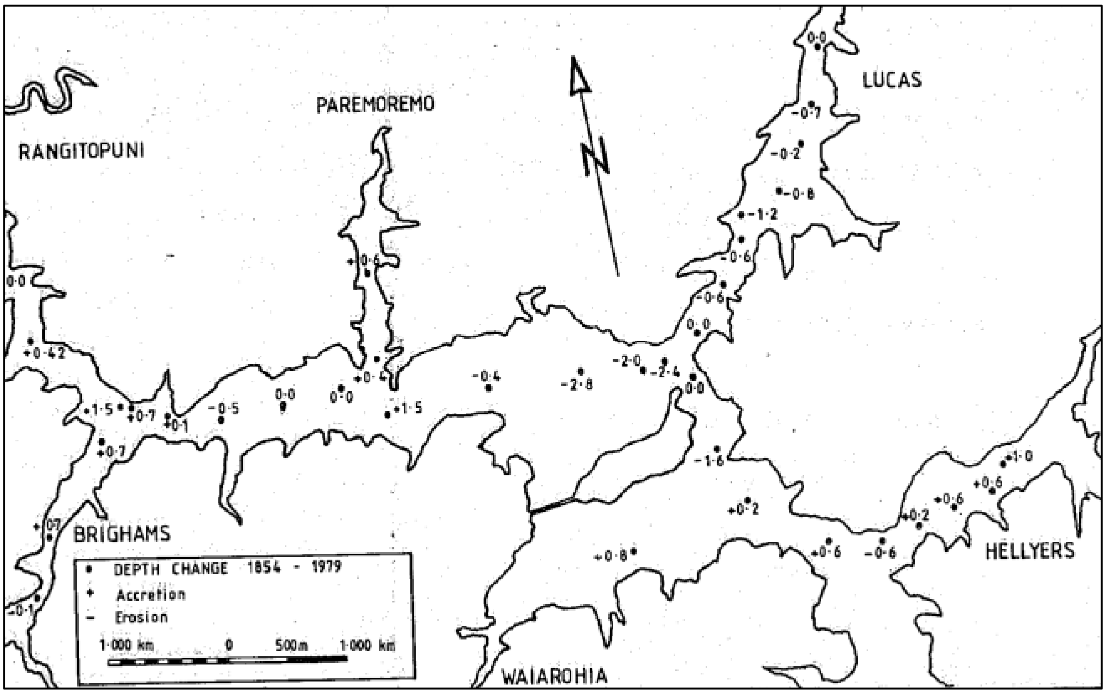


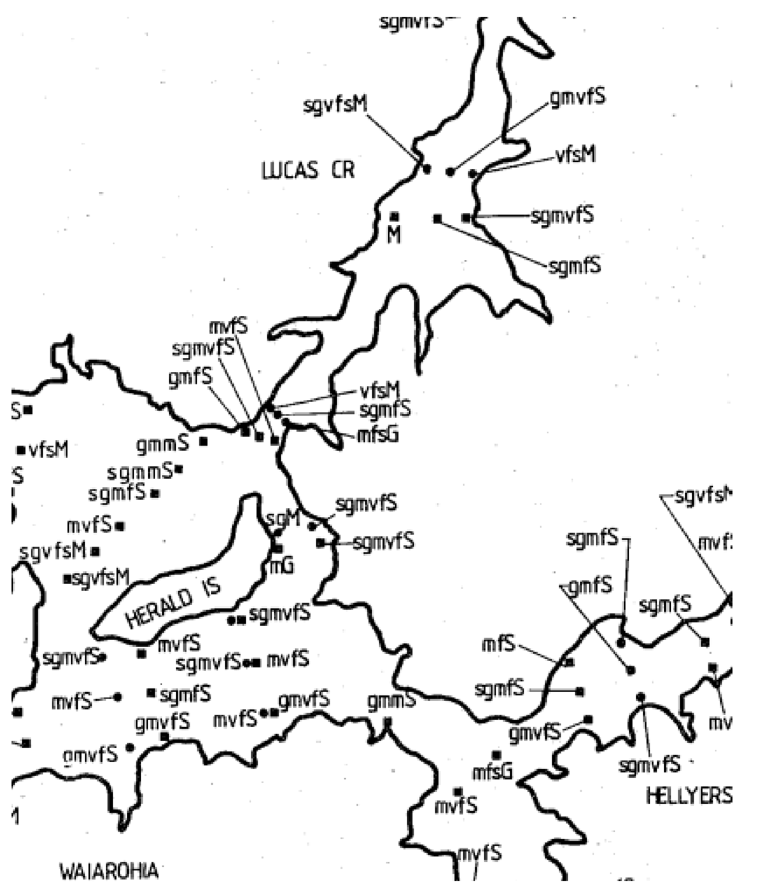

*Figure 3-3: Net sedimentation in the Upper Waitemata Harbour between 1854 and 1959 based on a comparison of soundings from the 1854 and 1979 bathymetric charts (Source: Hume, 1983)*

## 3.5 Sediments

Estuarine sediment dispersal and sedimentation are determined by the characteristics of the source material (size, shape, density, mineralogy and organic content) and the dynamics of the estuarine receiving waters. Water motions (principally tidal currents and associated turbulence) mix, disperse and, in places, re-suspend sediment particles. Gravity causes sediment to settle and deposit onto the estuary bed, but opposing this are the random movements of turbulent eddies, which increase in strength as current speed increases. The tidal creeks are the focus of freshwater–saltwater mixing, which promotes flocculation of freshwater-borne suspended sediments. This, in turn, increases particulate settling speed thereby promoting deposition of suspended particulate matter. Hence, deposition of terrigenous sediments and associated contaminants is favoured in tidal creeks (NIWA, 2004).

Sediment is generally sorted throughout the estuary by the ability of finer particles to be more easily transported by weaker currents. As a result, the finer particles are more likely to settle in very sheltered areas, such as the upper reaches of the tidal creeks and mangroves. Once deposited, fine sediment may dehydrate and consolidate, which makes it more resistant to subsequent erosion. Coarser particles, which can be moved only by the stronger currents that typically occur in channels, are more likely to remain within those channels. However, coarser particles are also found at the entrances to the creeks, where they have been deposited in the aftermath of large floods and where tidal currents are not energetic enough to re-suspend and re-disperse them (NIWA, 2004).

Sediment sampling was carried out during the investigations for the bridge duplication and additional investigations were carried out by Meritec (2002) and T&T (2014, 2015). Locations of the samples are shown in Figure 3-5 and results of surficial and near surficial sediment gradings are shown in Figure 3-6 and tabulated in Table 3-1 and Table 3-2.




**Terms**

**Major Component**

**M = Muds**

**S = Sands**

**G = Gravels**

**Secondary components**

s = slightly

f = fine

vf = very fine

m = muddy

g = gravelly

s = sandy

*Figure 3-4: Surficial sediment texture (Source: Hume, 1983)*

In the intertidal areas the sediment generally comprised fine sands (mean size range between 0.15 mm and 0.2 mm) although shells were evident in samples in the channel as evident by gradings larger than 2 mm, and a greater proportion of silts were observed in the intertidal flat adjacent to the causeway. The more silty sediments suggest lower energy environments where currents are weaker, enhancing settling.

The critical current and bed shear stress for current induced transport was calculated using the methods of Van Rijn (1990). Based on a D50 of 0.15 mm and a D90 of 0.4 mm the resulting critical velocity for initiation of movement for currents with no wave action was 0.64 m/s and the critical bed shear stress was 0.88 N/m$^2$. It is noted that wave action, particularly on the shallow intertidal areas when water depth is low, will initiate sediment transport at much lower tidal flows.

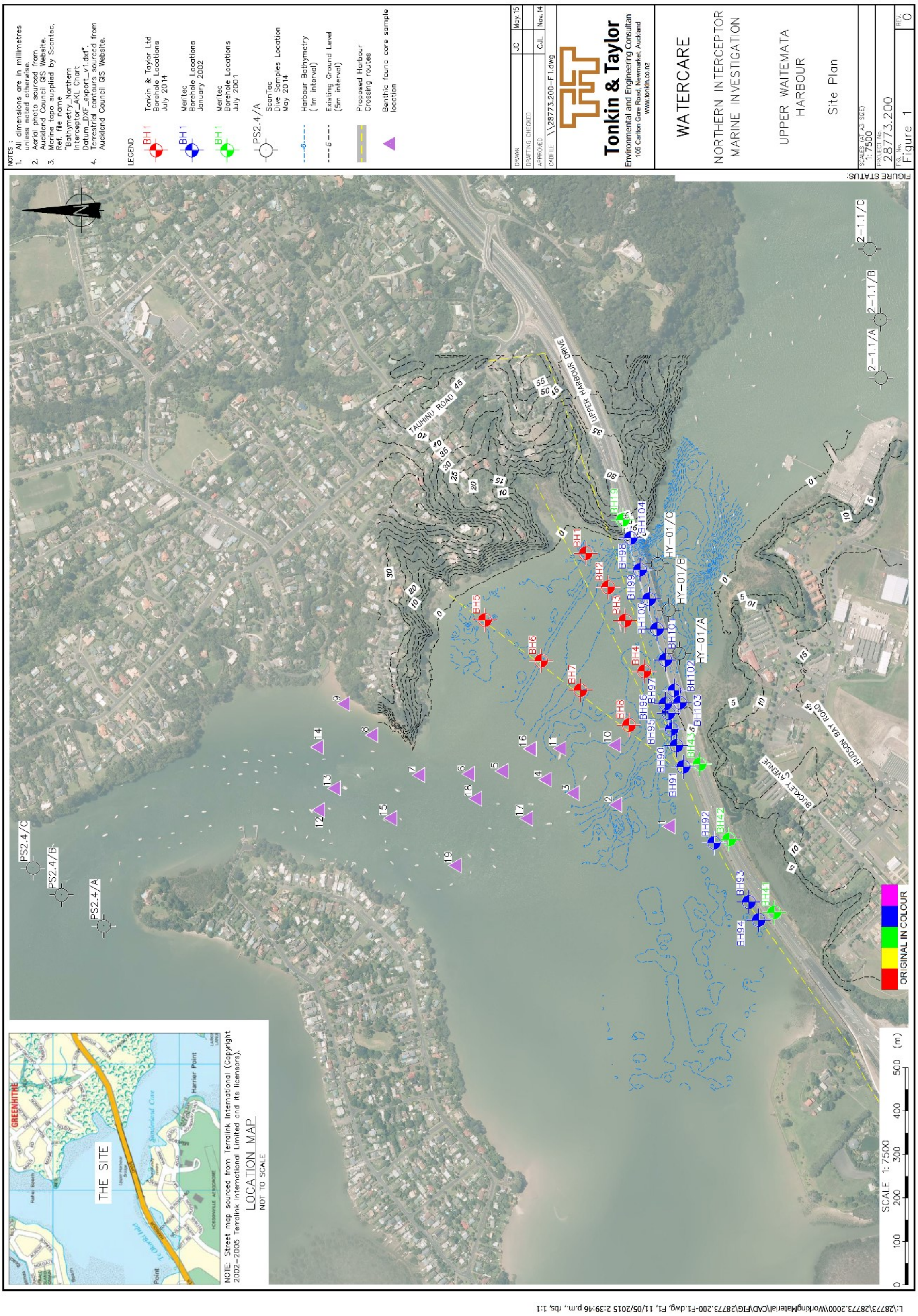



*Figure 3-5: Location of seabed testing*

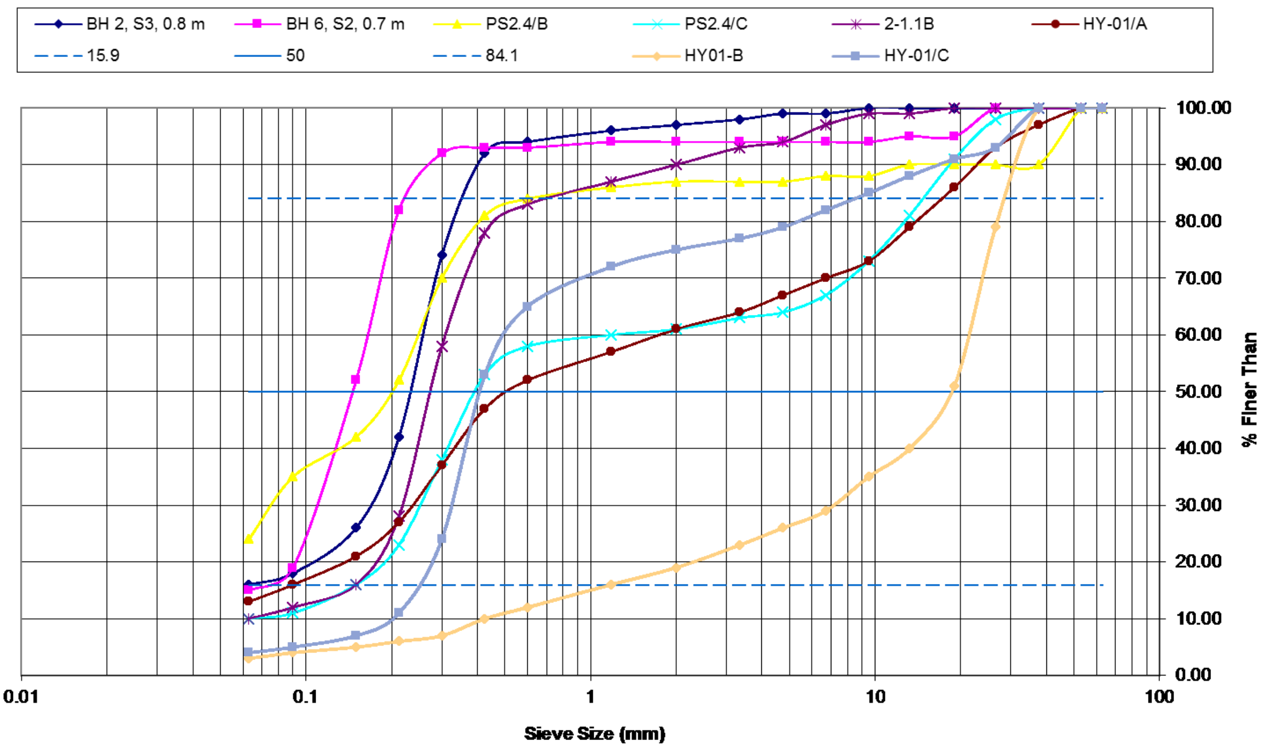



*Figure 3-6: Sediment grading curves for marine samples (historic sampling)*

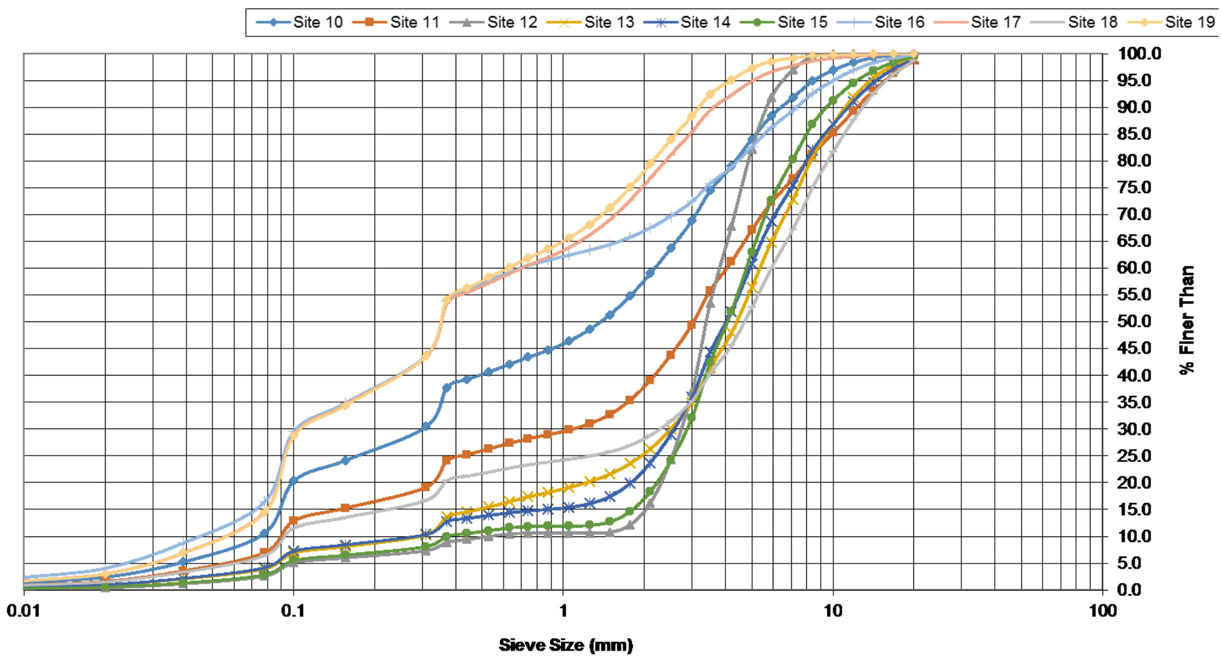



*Figure 3-7 Sediment grading curve for marine samples along alternative harbour crossing route*

**Table 3-1: Percent of sediment passing of marine samples from historic sampling**

| Sieve (mm) | BH 2, S3, 0.8 m | BH 6, S2, 0.7 m | PS2.4/B | PS2.4/C | 2-1.1B | HY-01/A | HY01-B | HY-01/C |
|---|---|---|---|---|---|---|---|---|
| <0.063 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0.063 | 16 | 15 | 24 | 10 | 10 | 13 | 3 | 4 |
| 0.09 | 18 | 19 | 35 | 11 | 12 | 16 | 4 | 5 |
| 0.15 | 26 | 52 | 42 | 16 | 16 | 21 | 5 | 7 |
| 0.21 | 42 | 82 | 52 | 23 | 28 | 27 | 6 | 11 |
| 0.30 | 74 | 92 | 70 | 38 | 58 | 37 | 7 | 24 |
| 0.43 | 92 | 93 | 81 | 53 | 78 | 47 | 10 | 53 |
| 0.60 | 94 | 93 | 84 | 58 | 83 | 52 | 12 | 65 |
| 1.18 | 96 | 94 | 86 | 60 | 87 | 57 | 16 | 72 |
| 2.00 | 97 | 94 | 87 | 61 | 90 | 61 | 19 | 75 |
| 3.35 | 98 | 94 | 87 | 63 | 93 | 64 | 23 | 77 |
| 4.75 | 99 | 94 | 87 | 64 | 94 | 67 | 26 | 79 |
| 6.7 | 99 | 94 | 88 | 67 | 97 | 70 | 29 | 82 |
| 9.5 | 100 | 94 | 88 | 73 | 99 | 73 | 35 | 85 |
| 13.2 | 100 | 95 | 90 | 81 | 99 | 79 | 40 | 88 |
| 19.0 | 100 | 95 | 90 | 91 | 100 | 86 | 51 | 91 |
| 26.5 | 100 | 100 | 90 | 98 | 100 | 93 | 79 | 93 |
| 37.5 | 100 | 100 | 90 | 100 | 100 | 97 | 100 | 100 |
| 53.0 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 63.0 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |

**Table 3-2: Percent of sediment sampling marine samples within alternative harbour crossing route**

| size (mm) | Site 10 | Site 11 | Site 12 | Site 13 | Site 14 | Site 15 | Site 16 | Site 17 | Site 18 | Site 19 |
|---|---|---|---|---|---|---|---|---|---|---|
| 20 | 99.9 | 98.8 | 100.0 | 99.3 | 99.1 | 99.5 | 99.8 | 100.0 | 98.8 | 100.0 |
| 16.8 | 99.7 | 96.3 | 100.0 | 97.9 | 97.2 | 98.4 | 99.2 | 99.9 | 96.2 | 100.0 |
| 14.1 | 99.3 | 93.1 | 100.0 | 95.5 | 94.6 | 96.8 | 98.3 | 99.8 | 92.6 | 99.9 |
| 11.9 | 98.4 | 89.3 | 100.0 | 91.9 | 91.1 | 94.5 | 96.9 | 99.6 | 87.7 | 99.9 |
| 10 | 96.9 | 85.2 | 100.0 | 86.9 | 86.9 | 91.3 | 95.0 | 99.2 | 81.7 | 99.8 |
| 8.4 | 94.9 | 81.2 | 99.4 | 80.8 | 81.9 | 86.9 | 92.6 | 98.7 | 75.1 | 99.6 |
| 7.1 | 91.8 | 76.7 | 97.0 | 72.8 | 75.5 | 80.3 | 89.4 | 97.8 | 67.2 | 99.2 |
| 5.9 | 88.4 | 72.3 | 91.9 | 64.9 | 68.8 | 72.6 | 86.2 | 96.6 | 60.0 | 98.6 |
| 5 | 84.1 | 67.1 | 82.2 | 56.4 | 60.8 | 63.0 | 82.7 | 94.9 | 52.7 | 97.3 |
| 4.2 | 79.0 | 61.2 | 67.7 | 47.9 | 51.9 | 51.9 | 78.9 | 92.3 | 45.6 | 95.1 |
| 3.5 | 74.5 | 55.8 | 53.4 | 41.5 | 44.3 | 42.4 | 75.8 | 89.5 | 40.4 | 92.5 |
| 3 | 68.9 | 49.5 | 36.9 | 35.0 | 35.9 | 32.1 | 72.5 | 85.5 | 35.2 | 88.4 |
| 2.5 | 63.7 | 43.8 | 24.2 | 30.0 | 29.0 | 24.1 | 69.7 | 81.1 | 31.5 | 84.0 |

| size (mm) | Site 10 | Site 11 | Site 12 | Site 13 | Site 14 | Site 15 | Site 16 | Site 17 | Site 18 | Site 19 |
|---|---|---|---|---|---|---|---|---|---|---|
| 2.1 | 59.0 | 39.1 | 16.2 | 26.3 | 23.7 | 18.3 | 67.5 | 76.8 | 28.8 | 79.4 |
| 1.77 | 54.9 | 35.4 | 12.2 | 23.6 | 19.9 | 14.7 | 65.8 | 72.7 | 27.0 | 75.1 |
| 1.49 | 51.3 | 32.8 | 10.8 | 21.6 | 17.4 | 12.7 | 64.5 | 69.1 | 25.8 | 71.3 |
| 1.25 | 48.6 | 31.0 | 10.6 | 20.2 | 16.1 | 12.0 | 63.4 | 66.2 | 25.0 | 68.2 |
| 1.05 | 46.4 | 29.8 | 10.6 | 19.1 | 15.4 | 12.0 | 62.5 | 63.9 | 24.4 | 65.7 |
| 0.88 | 44.7 | 29.0 | 10.6 | 18.2 | 15.0 | 12.0 | 61.5 | 62.1 | 23.9 | 63.6 |
| 0.74 | 43.4 | 28.2 | 10.6 | 17.4 | 14.8 | 11.9 | 60.5 | 60.6 | 23.4 | 61.9 |
| 0.63 | 42.1 | 27.4 | 10.5 | 16.5 | 14.4 | 11.6 | 59.3 | 59.0 | 22.7 | 60.2 |
| 0.53 | 40.7 | 26.3 | 10.0 | 15.5 | 13.9 | 11.1 | 57.7 | 57.3 | 22.0 | 58.2 |
| 0.44 | 39.3 | 25.3 | 9.4 | 14.6 | 13.4 | 10.5 | 56.0 | 55.5 | 21.2 | 56.3 |
| 0.37 | 37.8 | 24.1 | 8.9 | 13.7 | 12.8 | 10.0 | 54.0 | 53.5 | 20.4 | 54.1 |
| 0.31 | 30.5 | 19.2 | 7.3 | 10.3 | 10.3 | 8.1 | 43.7 | 43.6 | 16.7 | 43.7 |
| 0.156 | 24.2 | 15.3 | 6.0 | 8.1 | 8.4 | 6.5 | 34.9 | 34.7 | 13.5 | 34.5 |
| 0.1 | 20.3 | 12.9 | 5.1 | 6.9 | 7.2 | 5.5 | 29.6 | 29.0 | 11.6 | 28.8 |
| 0.078 | 10.6 | 7.0 | 2.6 | 4.0 | 4.1 | 2.8 | 16.4 | 14.6 | 6.4 | 14.5 |
| 0.039 | 5.2 | 3.6 | 1.2 | 2.1 | 2.2 | 1.3 | 8.8 | 6.9 | 3.4 | 6.9 |
| 0.02 | 2.4 | 1.6 | 0.5 | 0.9 | 1.0 | 0.6 | 4.0 | 3.0 | 1.5 | 3.0 |
| 0.0098 | 1.4 | 0.9 | 0.2 | 0.5 | 0.5 | 0.3 | 2.3 | 1.7 | 0.9 | 1.6 |
| 0.007 | 0.5 | 0.3 | 0.0 | 0.2 | 0.2 | 0.1 | 0.8 | 0.6 | 0.3 | 0.6 |

## 3.6 Suspended sediment concentrations

Suspended solids concentrations have been measured by the former Auckland Regional Council (ARC), Ports of Auckland Ltd (POAL), and NIWA at various locations around Waitemata Harbour. Analysis of these data is shown in Table 3-3. Based on average suspended sediment concentrations and the tidal flux, some 560 tonnes of sediment is transported each day within the tidal flows.

The sediment-settling rate of suspended material was measured at 1.5 m per hour from a site within the Port of Auckland (Beca, 1994) for a type of material, broadly referred to as marine mud. Settling velocities from sediment samples taken in Bayswater Marina found that the settling velocity of the fine sand fraction was of the order of 36 m/hour while the silt and clay particles settled at a slower rate of less than 3.6 m/hour. The median sediment size from the samples was 0.077 mm and the fraction of sand was 60% (KMA, 1988).

**Table 3-3: Suspended sediment concentrations within upper and middle areas of Waitemata Harbour**

| Location and period | Suspended sediment concentration (mg/L) |
|---|---|
| NIWA DOBIE gauge data (4 April 2006 to 8 June 2006), entrance to Catalina Channel | 20 (mean) |
| ARC data (April 1991 to May 1997), Chelsea | 12 (mean) |
| POAL data (August 1992 to October 1992), Westhaven Marina | 12 (mean) |

| Location and period | Suspended sediment concentration (mg/L) |
|---|---|
| POAL data (March 1995 to March 1997, Westhaven Marina | 9 (mean) |
| Fletcher Construction, Stage 1 dredging works, Bayswater Marina | 8 (mean) |
| Entrance to Upper Harbour (4 May to 27 July 2007) at 5.1 m mean water depth (Oldman et. al, 2008) | 1340 (max)<br>20 ($90^{th}$ percentile) |
| Hobsonville Jetty monthly data from 2003 (NIWA, 2004) | 7.2 (median), 4.9 (min), 25 (max) |
| Lucas Creek monthly data from 2003 (NIWA, 2004) | 13.4 (median), 8.3 (min), 41.9 (max) |

Measurements of suspended sediments on the fine sandy intertidal flats of the Manukau Harbour have identified that waves are the primary mechanism responsible for sediment re-suspension on estuarine intertidal flats at that location, with the highest suspended sediment concentrations occurring in the turbid fringe which occupies the shallow edges of the estuarine water body (Dolphin and Green, 1997). The measurements showed significant suspended sediment concentrations of 289 to 1640 mg/l) in shallow water depths even with small wave heights (less than 0.3 m). Due to the Upper Waitemata Harbour having similar flat sandy intertidal area as the Manukau Harbour these processes are also likely to occur along these intertidal areas.

## 3.7 Water levels

The tide follows a typical spring/neap tidal cycle, with spring range of 2.8 m and neap range of 2 m. Comparing the six-month tide record at the Salthouse Jetty in Lucas Creek at the end of Rame Road (Williams and Rutherford, 1983) with tides in the Commercial Harbour (Queens Wharf), shows that high tide occurs simultaneously at the two locations but is 0.15 m higher in the Upper Waitemata Harbour. Low tide is generally 0.12 m lower in the Upper Waitemata Harbour, and the time of low water is variable compared to Queens Wharf (+/- 20 minutes). This compares well with 0.14 m that was used to derive tide levels at Hobsonville Landing (Beca, 2009). Extreme water level information in the vicinity of the crossing is provided in NIWA (2013) coastal inundation report.

Water slopes within the Upper Waitemata Harbour are very small. The tidal prism (volume of the estuary at high water minus volume at low water) is $1.86 \times 10^7$ m$^3$ during spring tides and $1.00 \times 10^7$ m$^3$ during neap tides (NIWA, 2004).

## 3.8 Currents

Currents within the main body of the Upper Waitemata Harbour are driven principally by the tide and current patterns are governed mainly by local bathymetry (channels, mud banks, headlands, bays). It is only during large freshwater floods, and even then only in the tidal creeks, that freshwater momentum is a significant factor driving currents. Because the estuary is sheltered, wind typically does not generate strong currents (either as shear-driven surface movements or orbital motions associated with wind waves). However, wind-driven motions may become important under strong winds that blow down the long axis of the estuary (NIWA, 2004). The strong tidal influence and the elongate nature of the Upper Waitemata Harbour means circulation is largely parallel to the major channel systems. Exceptions occur where in the shallow water area north of Herald Island where anticlockwise rotatory circulations appear to be generated on flood flows. A variety of minor water movements occur, particularly over shallow intertidal areas due to wind generated currents and waves (Hume, 1983).

### Table 3-4: Tidal and extreme water levels within upper Harbour and Lucas Creek

| Event (%AEP/ARI) | Level (m AVD[3]) | |
|---|---|---|
| | Upper harbour | Lucas Creek |
| 0.5% AEP (200 yr ARI)[1] | 2.64 | 2.68 |
| 1% AEP (100 yr ARI)[1] | 2.59 | 2.62 |
| 2% AEP (50 yr ARI)[1] | 2.53 | 2.57 |
| 10% AEP (10 yr ARI)[1] | 2.41 | 2.45 |
| Highest Astronomic Tide (HAT)[2] | 2.0 (2.08)[4] | |
| Mean High Water Springs Perigean (MHWSP)[2] | 1.8 | |
| Mean High Water Springs 10 (MHWS10)[2] | 1.7 | |
| Mean High Water Springs Nautical (MHWSn)[2] | 1.6 (1.68)[4] | |
| Mean Low Water Spring Nautical (MLWSn) | -1.2 (-1.36)[4] | |
| [1]Extracted from Point 71 for Upper Harbour (1748618E, 5928117N) and 83 for Lucas Creek (1749079E, 5930540N) NIWA (2013)<br>[2]Estimated by adding 0.15 m to Queens Wharf Values from NIWA (2011)<br>[3]Auckland Vertical Datum (= Chart Datum +1.74 m)<br>[4]Tide levels established at Hobsonville by direct measurement (Beca, 2009) | | |

Tidal currents were measured using an Aanderaa RCM4 current meter situated mid depth in the Hobsonville Channel from 4-20 May 1982 in 8 m of water (NIWA, 2000). Higher velocities were observed on the ebb tide supporting the ebb scour hole as shown in Figure 3-2. Additional tidal current measurements were carried out by NIWA between 12 and 16 September 2002 using an S4 current metre deployed in 9 m water depth, 0.7 m above the seabed in the centre of the channel (809495.29N, 392340.49E, Mt Eden Circuit 2000) (see Table 3-5)

Currents are only slack (below 0.05 m/s) for 7% of the time when the tide is turning. The mean period of ebb tide is 5.5 hours compared with 6.9 hours for flood tide. The difference in tide periods explains the higher velocities recorded during ebb tides.

Additional tidal current measurements were carried out by NIWA between1 2 and 16 September 2002 using an S4 current metre deployed in 9 m water depth, 0.7 m above the seabed in the centre of the channel (809495.29N, 392340.49E, Mt Eden Circuit 2000).

### Table 3-5: Measured tidal currents in the Hobsonville Channel (Source: NIWA, 2000)

| Tide | Peak ebb velocity (m/s) | Peak flood velocity (m/s) |
|---|---|---|
| Spring tide | 0.70 | 0.58 |
| Neap tide | 0.42 | 0.30 |

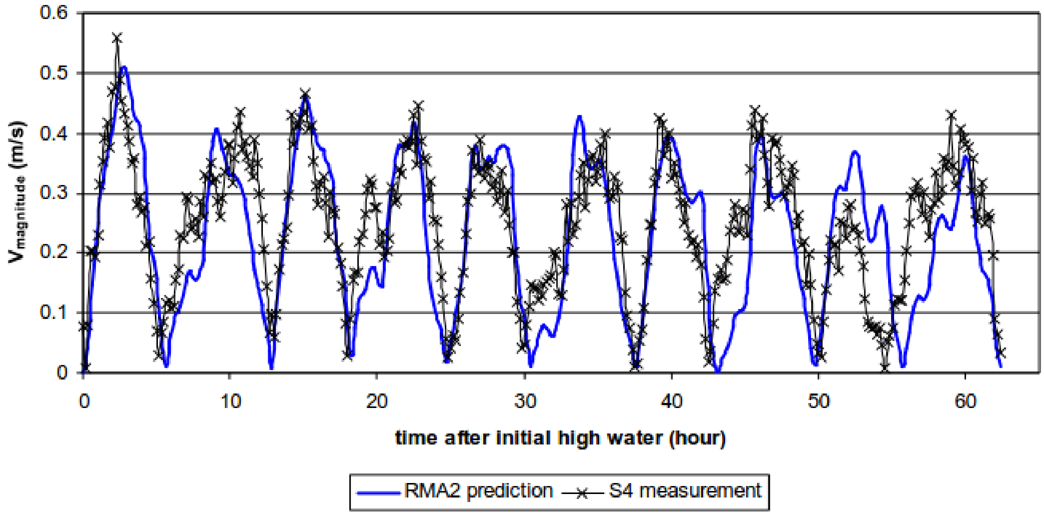
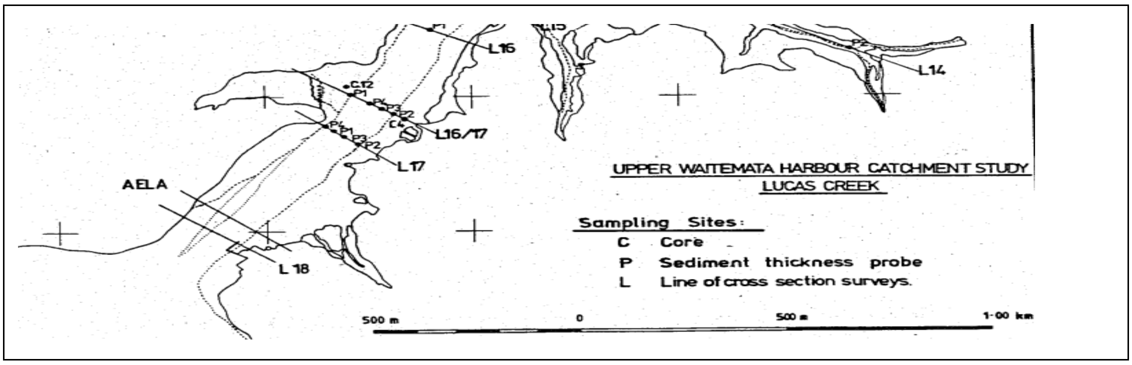



*Figure 3-8: Comparison of measured and predicted current velocity in the Hobsonville Channel between 12 and 16 September 2002 (Source: Beca, 2009)*

Williams and Rutherford (1983) measured mid-channel tidal currents up to 0.7 m/s during peak ebb tide in the lower reaches of Lucas Creek and McLachlan and Hume (1981) measured spring tide measurements across Lucas Creek on 11 March 1981 (refer Figure 3-9).

A computational model of the Whau River, Central Waitemata Harbour and Waterview Estuary was constructed using the MIKE 21 FM coastal model. This model was extended to include the Upper Harbour area, Lucas Creek, Rangitopuni Creek and Rarawara Creek (refer Figure 3-10 for full model extents) to provide better representation of the upper harbour. The model grid was refined around the causeway to provide greater resolution (refer Figure 3-11). Details of the model set-up and verification are included in Appendix B. This computational model suite is an industry accepted model for simulating coastal and inland flows. It has previously been successfully applied to several coastal areas of New Zealand, including the Waitemata Harbour.



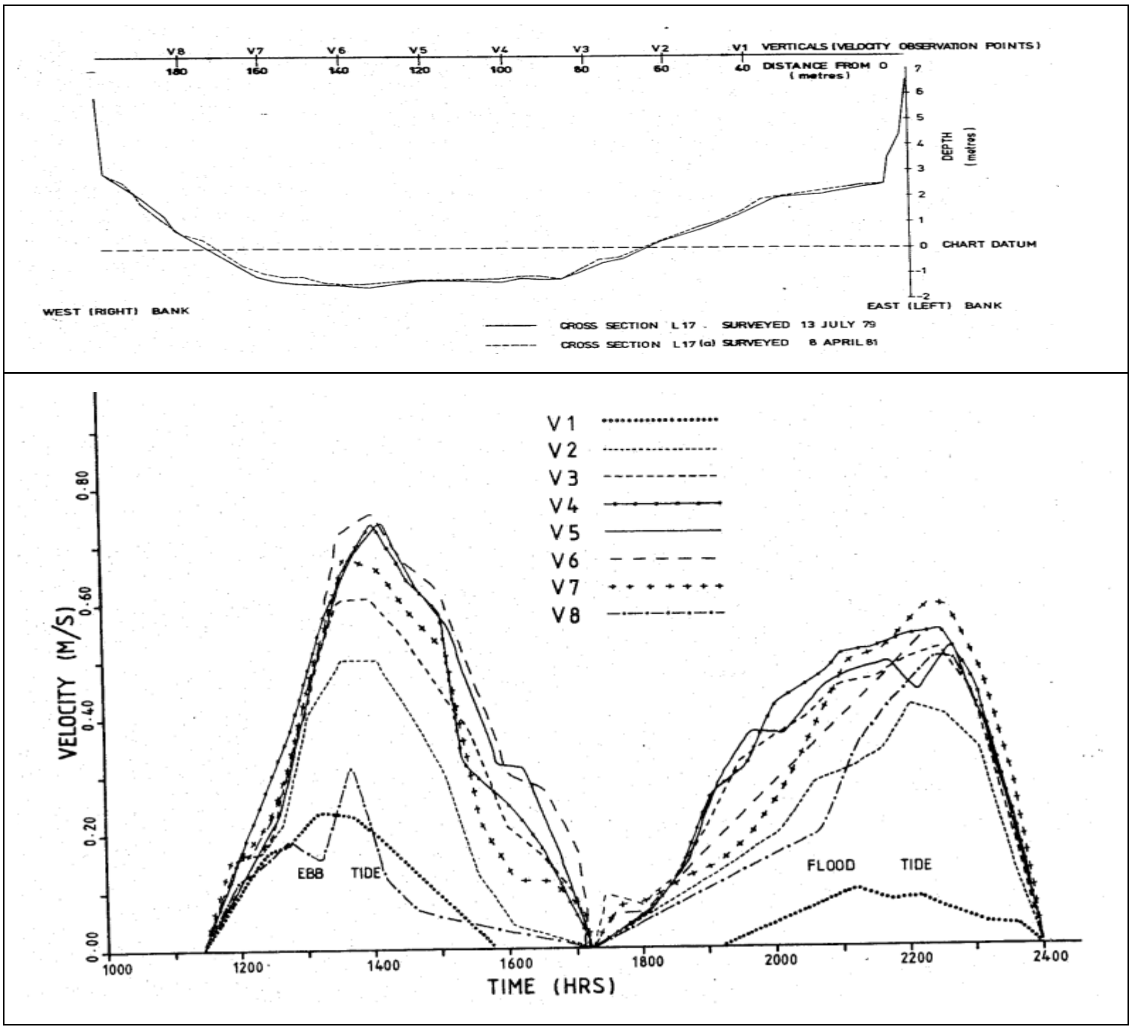



*Figure 3-9: Spring tide velocity-time profiles on 11 March 1981 (Source: Hume 1983)*

The modelling result of maximum flood and ebb spring tides are shown in Figure 3-12 and Figure 3-13 and modelled peak currents in the centre of the channel included in Table 3-6. There results show highest currents are confined to the main channel. Typical maximums are in the order of 0.5 m/s for flood and ebb spring tides, consistent with historic observations of Beca, but slightly lower than the observations of NIWA.

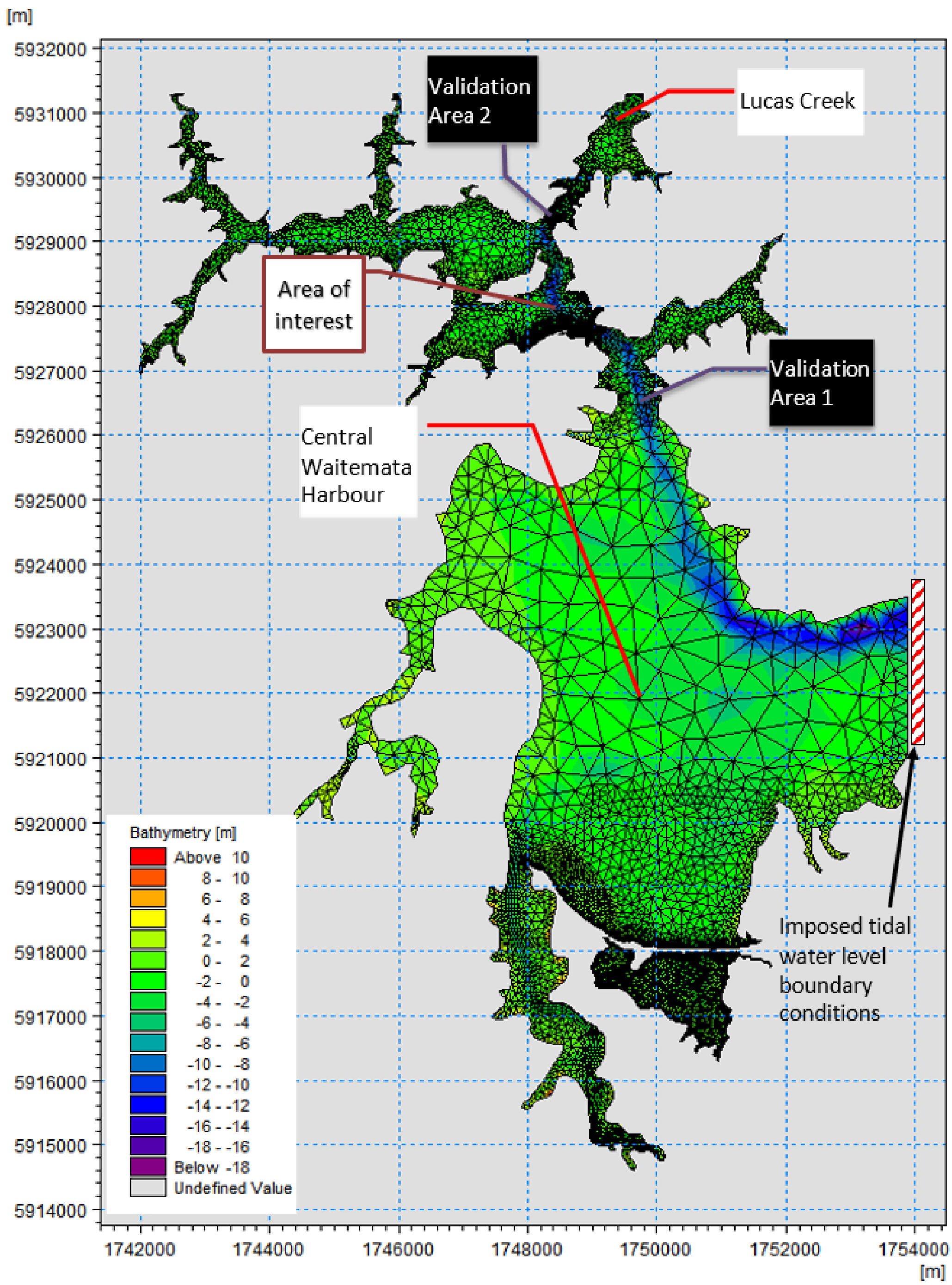



*Figure 3-10: Extent of hydrodynamic model*

The results show highest currents in the main channel adjacent to the Greenhithe shoreline (greater than 0.8 m/s) and lower currents are present on the intertidal areas, although slightly higher ebb velocities (up to 0.3 m/s) are present than occur during flood tides (up to 0.2 m/s) due to the sheltering nature of the causeway during flood conditions compared to ebb conditions.

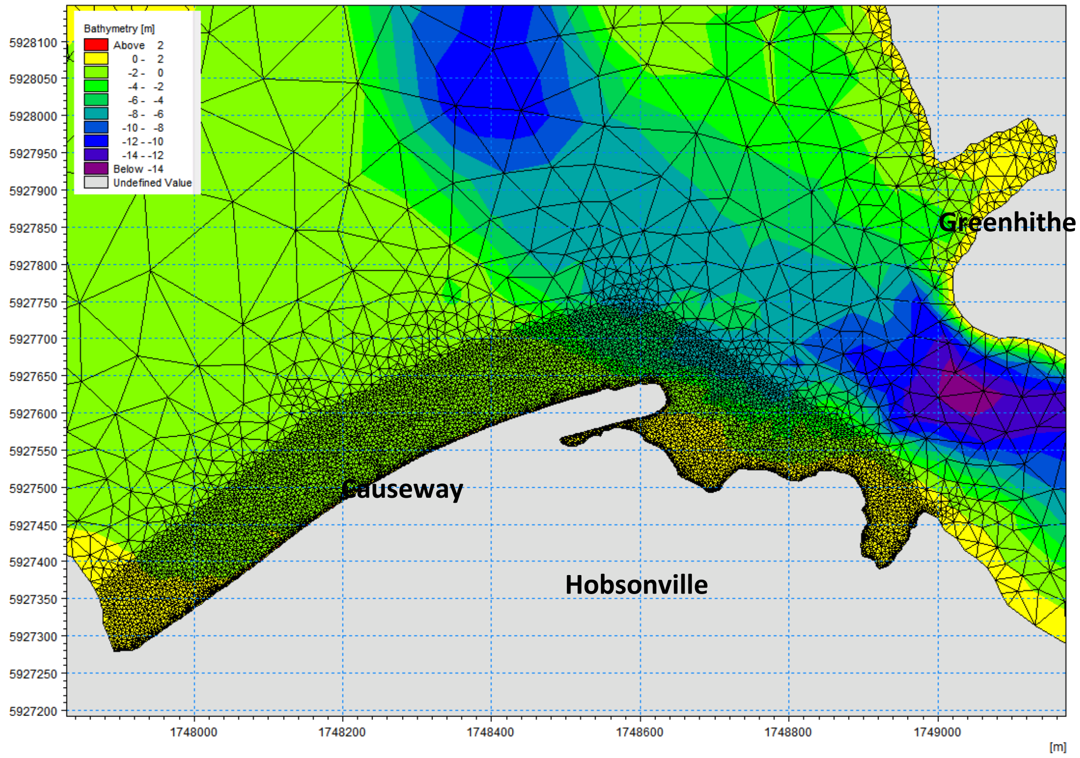



*Figure 3-11: Detailed modelling grid around main tidal channel and existing causeway*

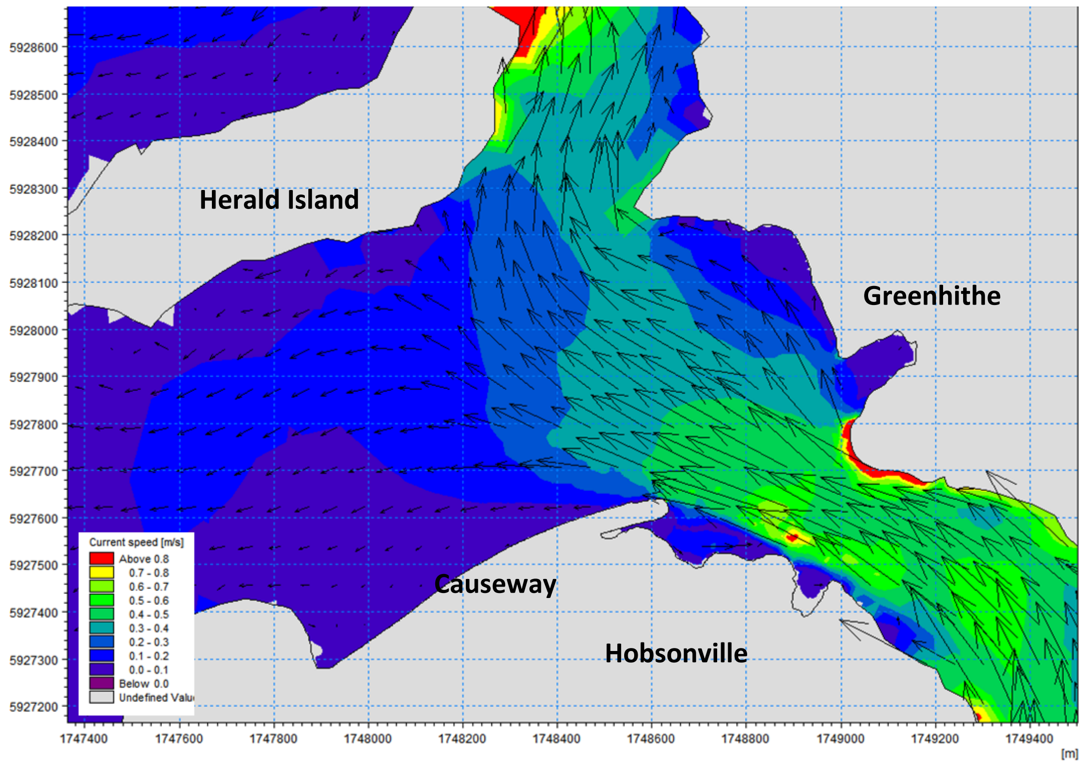



*Figure 3-12: Spring tide – maximum flood velocity*

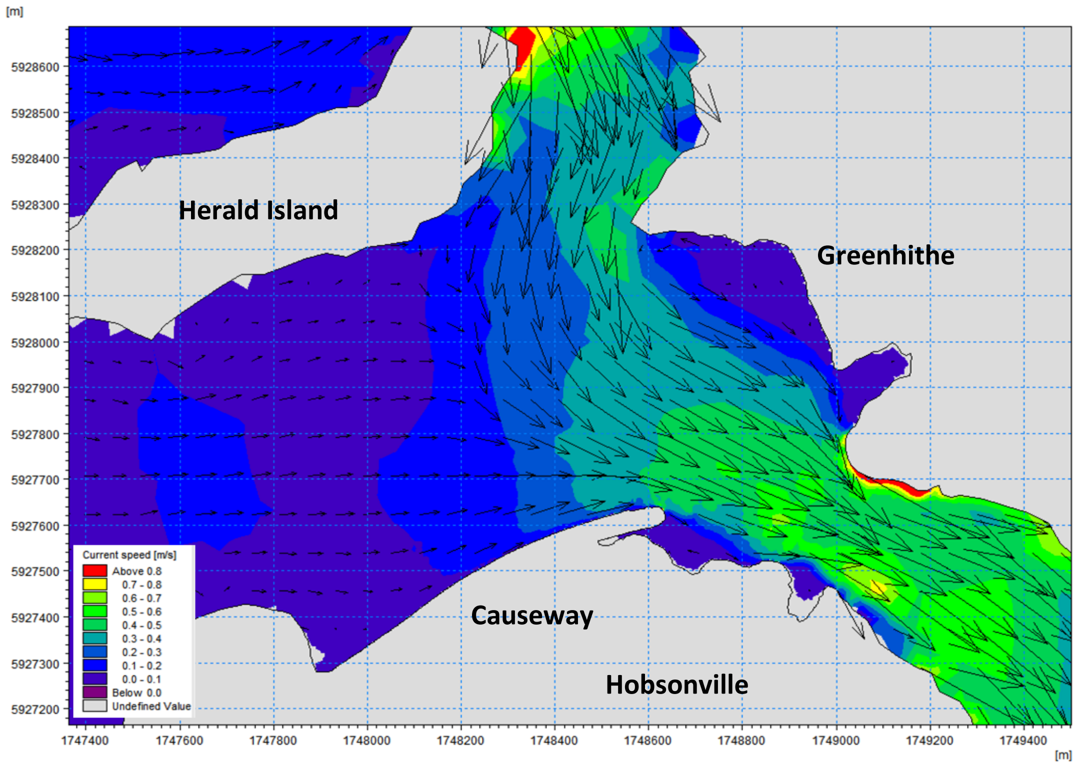



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*Figure 3-13: Spring tide - maximum ebb tide velocity*

**Table 3-6: Modelled tidal flows in main channel under existing bridge**

| Tide | Peak flood velocity (m/s) | Peak ebb velocity (m/s) |
|---|---|---|
| Springs | 0.497 | 0.464 |
| Neaps | 0.244 | 0.251 |

## 3.9 Winds

Wind data has been obtained from the Whenuapai Aero club using NIWA's Cliflo website. This site provides some 52 years of daily observations (refer Figure 3-14). While strong wind speeds of around 20 m/s can occur from all directions, predominant wind directions are from the west-north west to the south west. The maximum recorded wind speed of 22.6 m/s originated from the north east.

## 3.10 Waves

Waves at the upper harbour area between Hobsonville and Herald Island are fetch and depth limited, with maximum wave heights being generated at high water levels with winds from either the south west to north east (for the causeway area) and less frequently from south-east along the main channel (for the bridge). Based on standard empirical relationships of Wilson (Goda, 2003) the significant wave height and period has been determined for a 100 year return period wind speed at MHWS.

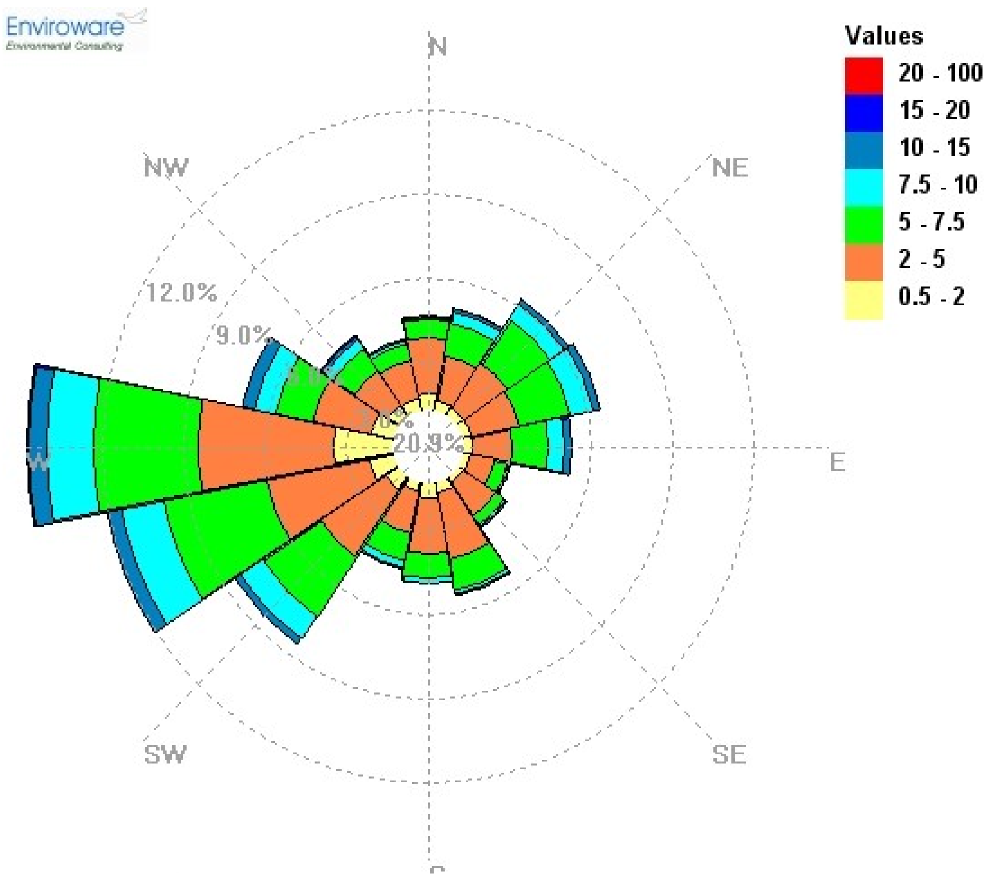


*Figure 3-14: Wind rose data from Whenuapai Aero Club, Jan-1960 to Dec 2012 (Source: Cliflo)*

**Table 3-7: Extreme nearshore wave heights at MHWS tide level within Upper Waitemata Harbour between Hobsonville and Herald Island**

| Wind Direction | Fetch (km) | Average fetch depth at MHWS (m) | Significant wave height, Hs (m) | Significant wave period, Ts (s) |
|---|---|---|---|---|
| W | 1.5 | 2.8 | 0.71 | 2.3 |
| NW | 0.8 | 2.8 | 0.55 | 1.9 |
| N | 1 | 7.8 | 0.65 | 2.1 |
| NE | 0.8 | 6 | 0.55 | 2.0 |

Based on depth limited empirical relationships, maximum significant wave heights of between 0.5 m and 0.71 m are possible at this location, with periods of between 2 and 3 seconds along the causeway. During south easterly conditions wave heights could reach up to 1.0 m within the channel with periods of around 3.4 seconds (Beca, 2009).

## 3.11 Coastal processes

The Upper Waitemata Harbour area bounded by Hobsonville Point, Greenhithe and Herald Island is a relatively low energy environment dominated by tidal flow concentrations through the narrow channel and wind generated wave conditions on the intertidal flats. As shown in the comparison of bathymetric charts (refer Figure 3-3) the intertidal flats are likely to be depositionary areas. Historic studies have indicated long term sedimentation patterns in sheltered areas, particularly within the numerous inlets and creeks (Hume, 1983) although in the more exposed areas, subject to higher wave and tidal flows, there is no clear trend.

An analysis of aerial photographs was carried out by NIWA (2000) using photographs from 1972 (pre original bridge construction) and 2000. Oblique aerials from Whites Aviation are available from 1949 to 1973 (refer Appendix C) and additional aerial images are available from 1959, 1996, 2006 and 2008 and satellite images via Google Earth for 30 August 2004, July 2006 to March 2007, 28 March 2009 and 30 August 2013 (refer Appendix D). It is noted that these images are at different stages of the tide, but they do provide a means of assessing global changes. The original causeway construction can be seen in Figure C-6 (Appendix C) and the causeway widening and duplication of the upper harbour bridge is shown in Figure D-3 and D-4 (Appendix D).

The NIWA (2000) assessment identified a slight retreat of the intertidal flat on the southern side of the western bridge abutment that was attributed to the higher flow velocities from the construction formed by the abutment. Additional impacts identified by NIWA included mangrove growth and the development of drainage channels associated with the drainage works through the causeway.

The results of our review of the available photographs, particularly since the duplication of the bridge was completed, show less than minor changes in coastal processes with changes mainly associated with the increase in mangrove growth. The retreat of the intertidal flat on the southern side of the western bridge abutment has not continued. Even from the 2008 high resolution photograph (Figure D-6, Appendix D) the shoal that formed immediately to the south of the abutment has progressively become covered with mangroves. Comparing the 1996 photograph (Figure D-2) with the 2006 aerial (Figure D-4) and the 2013 satellite image (Figure D-8) which appear to be at similar states of the tide, show little net change over this 17 year period.

The only other change evident from the higher resolution photographs available since NIWA's study is the migration of the shell ridge that extends south east of the private land area at the western end of the causeway. It has moved more landward from 1959 to 1999 although has been relatively stable since that time. It is assumed that the causeway modified catchment flows in this area and enabled the shell ridge to migrate landward. However, from 1999 it has appeared to be relatively stable to accretionary.

## 3.12 Climate change and sea level rise

Historic sea level rise in New Zealand has averaged 1.7 ± 0.1 mm/year (Bell and Hannah, 2012). Climate change is predicted to accelerate this rate of sea level rise into the future. NZCPS (2010) requires that the identification of coastal hazards includes consideration of sea level rise over at least a 100 year planning period. Potential sea level rise over this time frame are likely to significantly alter the coastal hazard risk.

The Ministry of Environment (2008) guideline recommends a base value sea level rise of 0.5 m by 2100 (relative to the 1980-1999 average) with consideration of the consequences of sea level rise of at least 0.8 m by 2100 with an additional sea level rise of 10 mm per year beyond 2100. Bell (2013) recommends that for planning to 2115, these values are increased to 0.7 and 1.0 m respectively. Bell (2013) also recommends that when planning for new activities or developments, that higher potential rises of 1.5 to 2 m above the present mean sea level should be considered to cover the foreseeable climate-change effects beyond a 100 year period.

Modelling presented within the most recent IPCC report (AR5; IPCC, 2014) show predicted global sea level rise values by 2100 to range from 0.27 m, which is slightly above the current rate of rise, to 1 m depending on the emission scenario adopted. Extrapolating the RCP8.5 scenario to 2115 results in a sea level range from 0.27 to 0.47 m by 2065 and 0.62 to 1.27 m by 2115 (Figure 3-15). The RCP8.5 scenario assumes emissions continue to rise in the 21st century. Adopting this scenario is considered prudent until evidence of emission stabilising justify use of a lower projection scenario.

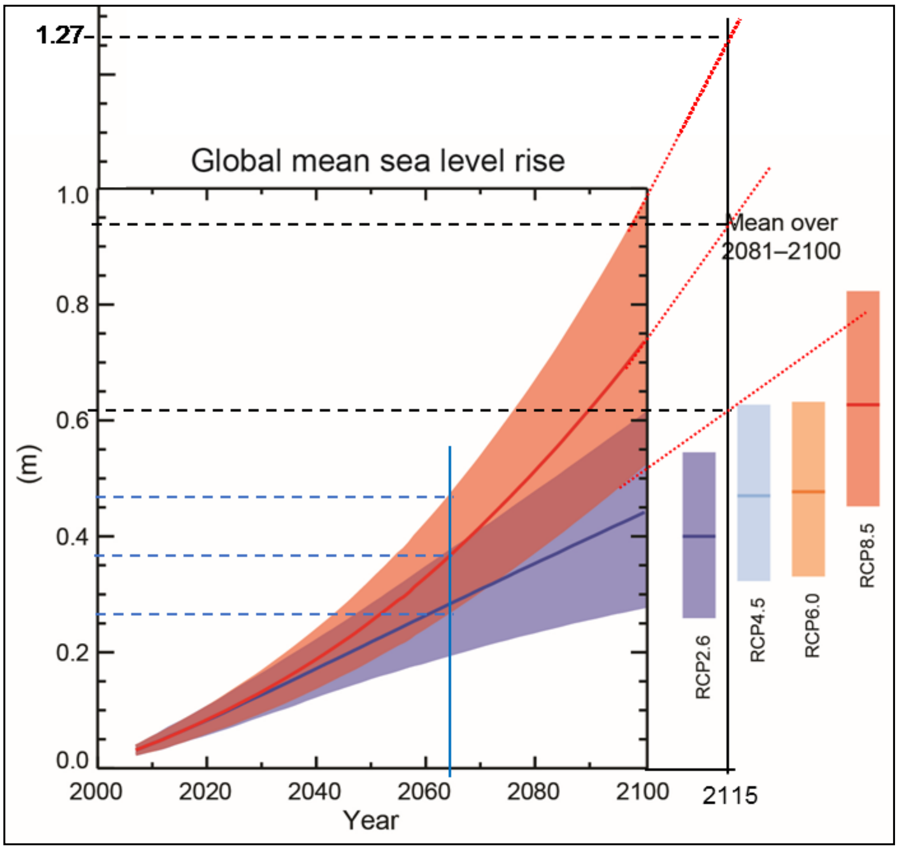


*Figure 3-15: Projections of potential future sea level rise presented within IPCC AR5 (IPCC, 2014) with adopted values for this assessment at 2065 and extrapolated to 2115*

# 4 Description of activity in CMA

The components of the work that potentially impact on coastal processes is the dual harbour pipeline that extends from the causeway of the western abutment of Upper Harbour Bridge to the reserve at the end of Rahui Road, Greenhithe and the crossing of Te Wharau Creek.

## 4.1 Construction approach

A complete description of the construction methodology is included in the AEE, although it is noted that the selected construction technique may change as a result of contractor preference.

### 4.1.1 Upper Waitemata Harbour crossing

The pipes crossing the harbour will either be installed by horizontal directional drilling (HDD) or by marine trenching. Due to the differing requirements of both methods, there are different alignments proposed for these options. The different alignment options are shown in Figure 4-1. This figure shows the expected trench area (the solid parallel lines) and the wider area where the pipe route could be situated.

For HDD the alignment is relatively straight while for the marine trenching method the alignment is predominantly along the main channel alignment, with connections to land across the intertidal area. Depending on the final alignment chosen, the marine trenching option may initiate from the northern end of the extended causeway. In this instance there would be no intertidal trenching adjacent to the causeway.

Due to the shallow nature of the harbour, separate construction methodologies have been developed for the marine trenching option for the intertidal areas and the installation of the pipeline across the main channel, with specific consideration of the interfacing between these two areas to connect the pipe.

### 4.1.2 Horizontal directional drilling

In this option all works will be carried out on established land areas on the widened causeway and at Rahui Road. The drilling will locate the pipeline in competent ground below the seabed level. Key risks of this methodology is accidental and uncontrolled spillage of drill water into the CMA. It is assumed that methods to prevent this likelihood will be included in a Construction Management Plan (CMP), prepared by the selected tenderer.

### 4.1.3 Marine trenching

#### 4.1.3.1 Subtidal area

The length of the pipeline route in the subtidal area is around 1170 m and will have a relatively narrow occupation width as they will be connected by a common ballast block. In this location the pipes may not be backfilled, so consideration is given both to the requirements if backfilling is carried out and if it isn't.

Carrying on a similar method of trenching as proposed for the intertidal area in the subtidal area is unlikely to be practical as there will be localised transport of the excavated sediment and increased risk of trench infilling during the construction period. Alternative methods such as a jet trenching (fluidisation), or a mass flow excavation using low head fan unit could be used. These devices are towed by a barge or tug will be used to either sink the pipes that are lain on the existing surface through the marine sediments to the required depth or to form a trench in which the pipeline can settle.

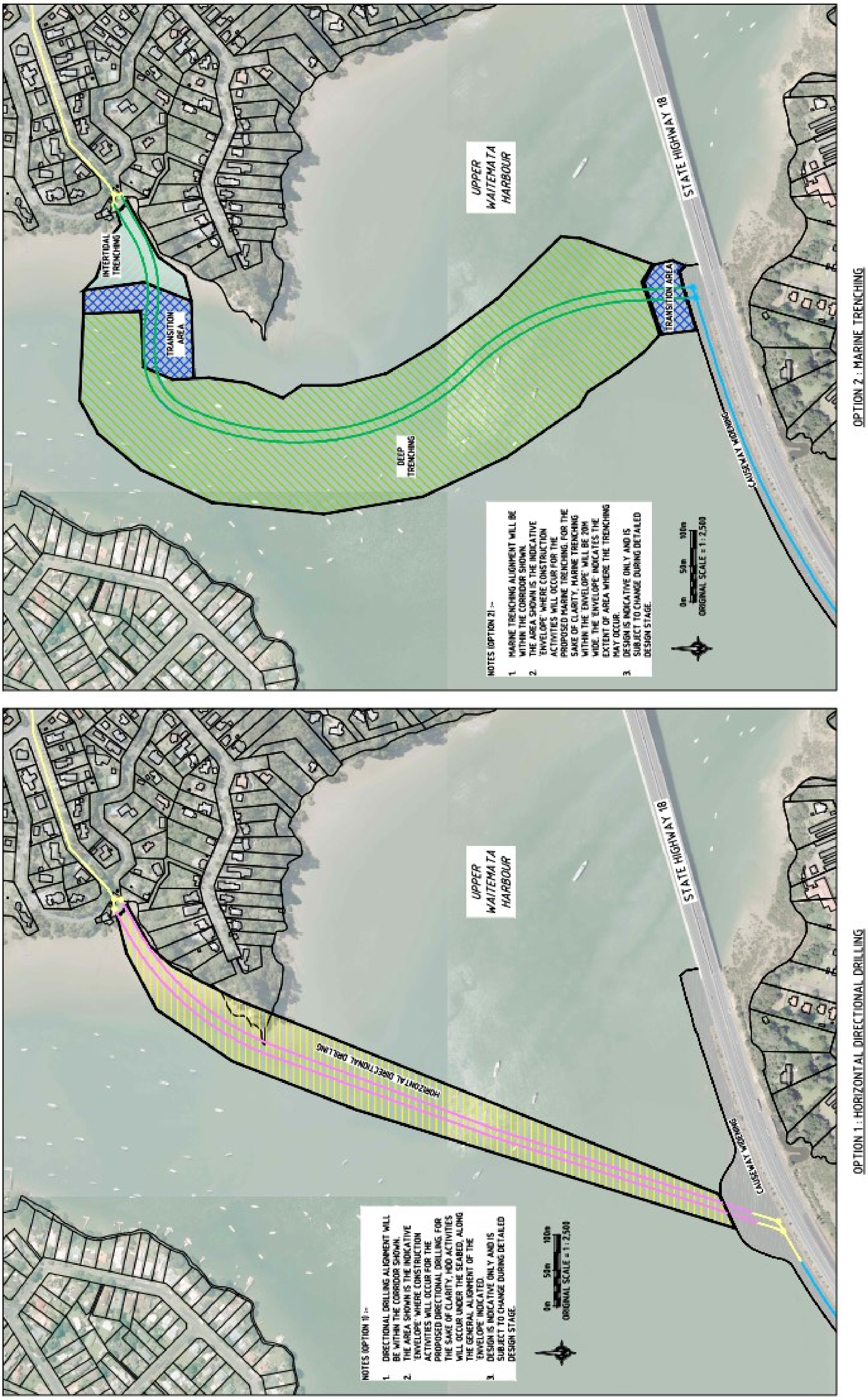


*Figure 4-1 Alternative routes for crossing Upper Waitemata Harbour*

**Mass flow excavation**

Mass flow excavation is a non-contact method for controlled flow excavation of trenches. The pipeline is placed on the existing seabed with the mass flow excavator then travelling along the length of the pipe forming the trench the pipe settles into. Alternatively the trench can be formed first with the pipe placed in subsequently. These devices typically travel at around 180 metres/hr and the base of the jet operates at 1 m above the seabed. The device draws in seawater and jets water out from the vertical down pipe at velocities of up to 10 m/s (flows up to 4 $m^3$/sec). The bed material is shifted and trenched with the force of the jet and flushed away.

Due to the reasonable tidal flows, the trench formed by this process is unlikely to be quickly backfilled by natural forces, but may gradually infill over time. Depending on how the excavated sediment settles, some of the excavated sediment could be used for manual backfilling of the trench (if required). Alternatively, other forms of backfilling may be required, which would require additional sediments or granular fill to be imported and placed on the seabed. This is likely to be done by barge mounted hydraulic excavators or a barge mounted shute.

**Jet trenching (fluidization)**

Trenching by fluidization is achieved by using jets of water to displace sediment from the trench (refer Figure 4-2). Jet trenching machines can be operated from small shallow draft barges and are therefore ideal for shore approaches in very shallow water. They are suitable for both sands and clays up to 190 kPa undrained shear strength. For economic reasons, they are suitable for short pipelines (less than 30 km long) and for large pipelines (>760 mm OD) where trenches deeper than 1.7 m are required (http://www.oes.net.au/optc_pipelines.shtml).

In this instance the pipeline is lain along the existing seabed. The jets can operate on sledges or travel along the pipeline route. Due to the requirement to install two pipes, it is likely that a machine that straddles the pipeline will be preferred over a sledge, as the sledge would need to span a significantly greater width of the seabed to avoid stability issues resulting from the disturbed seabed and this would increase the size and mass of the machine and hence the size and draft of the vessel required to tow the machine.

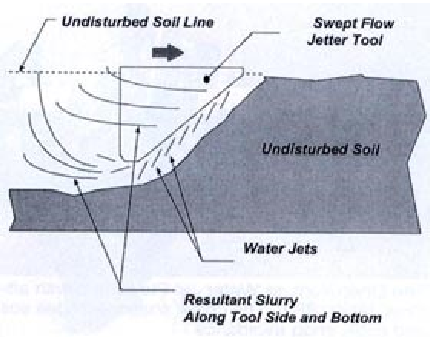


*Figure 4-2: Schematic of jet fluidization process (Source: Kober et al, 2001)*

An example of a jet trenching machine that travels along the pipeline is shown in Figure 4-3. (Machine are typically 3-6 metres long, weigh 3-9 tonnes and can be operated in deeper water from a small vessel). This option would effectively self-bury the pipe and no additional backfilling would be required.

*Figure 4-3: Example of jet trench machine (source: [http://www.oes.net.au/photographs_projects_and_versatility.shtml](http://www.oes.net.au/photographs_projects_and_versatility.shtml))*

#### 4.1.3.2 Transition areas

The transition areas connect the intertidal area with the subtidal area and typically include the edge of the main channel (refer Figure 4-1). In these areas the pipe will be installed either using hydraulic excavators mounted on a barge to form the trench or mobile hydraulic jetting techniques to self-lower the pipe lain out on the existing seabed as described in Section 4.1.3.1. In the case of the hydraulic excavator method, excavated material will either be stockpiled in a barge for replacement within the dredged channel or located on the seabed adjacent to the trench for backfilling.

#### 4.1.3.3 Intertidal area

The intertidal area is the sand flat extending from the edge of the main channel, that is, to around -1.3 m RL to landfall at Rahui Road (refer Figure 4-1). Due to the intermittent and shallow water depth present over the intertidal area it is not possible to carry out marine based dredging as there is insufficient navigable draft. Therefore, conventional land based methods to form the trench, with works carried out at low tide periods, is the most practical solution to form a dredged channel in this area.

To minimise effects, the trench is proposed to be perpendicular to the shore and is around 150 m long. To provide for 0.3 m of cover after the pipes are installed, the excavated trench will need to be around 1.0 m to 1.5 m deep.

The excavated sands and sediments will either be placed adjacent to the excavated trench for use in backfilling once the pipe is installed, or it will be trucked off-site and stockpiled for subsequent backfilling of the trench. Due to the shallow depth of the trench in the intertidal area shoring, or other treatment of the edges to the excavation, is unlikely to be required to reduce side slope slumping. In areas that trenching will extend into weathered rock the backfill will be graded armour rock.

The temporary construction access bund is likely to comprise coarse gravels overlying a geotextile separation layer and depending on the ground conditions a geo grid may also be used. The geotextile separation layer will facilitate the removal of the gravels and reduce contamination of this material with the existing seabed.

The temporary construction access bund is likely to have a crest elevation of around 2 m RL to be above the typical high tide level, resulting in a height above the existing seabed of around 0.5 to 1.0 m and a crest width of around 3.0 m.  This is a similar detail to the temporary causeway bund used in Hobson Bay which is shown in Figure 4-4.  The temporary construction access bund will be removed after construction is complete.


*Figure 4-4: Temporary causeway in Hobson Bay*

### 4.1.4 Te Wharau Creek

The works along the foreshore areas of Te Wharau Creek do not affect the creek directly with HDD techniques proposed to install the pipes in competent ground under the creek bed.

# 5 Assessment of effects

This section assesses the potential effects of the proposed activities and includes consideration of the construction effects (short term) and longer term effects on the physical coastal processes operating at this location.

## 5.1 Potential effects

The dredged channel will disturb the seabed during the construction process and the effect of this construction activity is considered in this assessment of effects. Depending on the nature of the excavated sediment there may be some effects over the longer term if sediment properties of the replaced sediment are different from the surficial sediments adjacent to the dredged area.

The assessment of effects addresses the potential effects of the permanent works (long term effects) as well as potential effects of the shorter term construction activity as it relates to the physical coastal processes.

### 5.1.1 Subtidal trenching across tidal channel

As discussed in 4.1.3.1 conventional trenching in the channel is not readily feasible and hydraulic jetting or similar processes are preferred. The potential effects of this approach can be evaluated by reference to a case study of this option recently completed in the USA.

#### 5.1.1.1 Case study

A Study on the effect of trenching a 660 mm pipeline by hydraulic jetting have been carried out in the New York Bight, USA (HDR, 2013). The requirement was to establish some 3.2 km of pipeline 2 m below the seabed over a cross-sectional area of 15.1 $m^2$. In this situation trenching rates of around 180 m/hr were typical although rates of between 122 m/h and 366 m/hr were possible. Sediment laden water discharges for the typical production rate were around 70 litres per minute.

This study provides useful comparison of potential effects to what might occur in the Upper Waitemata Harbour as the physical environment is reasonably similar:

- It is an estuarine area with water depth varying from 1 m to 30 m below mean low water compared to 1 to 15 m along the proposed pipe route
- Sediment comprising fine sands (D50 around 0.15 mm) which is generally representative of the minimum envelope of sediment size within the Upper Waitemata Harbour
- The tide range is around 1.5 m and peak tidal currents varied from around 0.2 m/s to 0.4 m/s compared to around 2.2 m and currents ranging between 0.25 m/s to 0.5 m/s within the main channel
- Insitu sediments and a natural sedimentation rate of around 2.5 mm/year
- Typical suspended solid concentrations of between 3 and 18 mg/L with a mean of around 5 mg/L compared to a recorded mean within the Upper and Central Waitemata Harbour of around 8 mg/L.

A 3D hydrodynamic model study was carried out to investigate suspended sediment concentrations near the seabed, with the water column divided into five layers.

The following series of figures show the resulting suspended sediment concentration at the bottom layer for the modelled worst case scenario of a rate of 366 m/hr as the dredging progresses (Figure 5-1 to Figure 5-5) and the resulting maximum suspended sediment concentration at each water depth layer for the entire construction period (Figure 5-6).

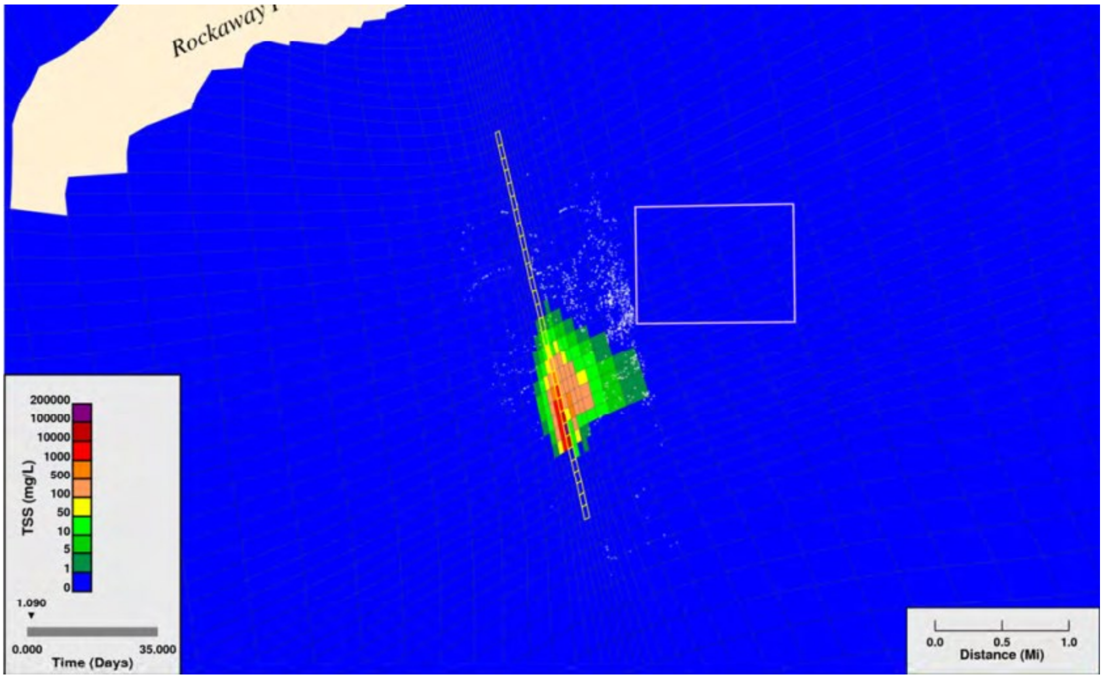


*Figure 5-1: Bottom layer projected solids concentration with rate of 366 m/hr, trenching 25% complete (Source: HDR, 2013)*

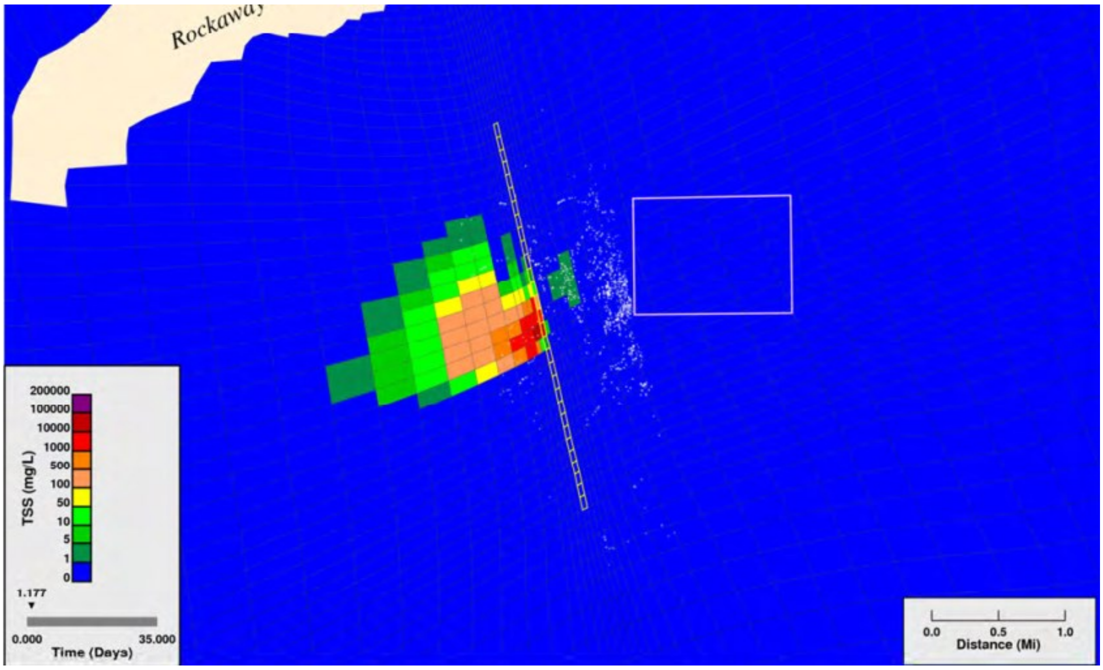


*Figure 5-2: Bottom layer projected solids concentration with rate of 366 m/hr, trenching 50% complete (Source: HDR, 2013)*

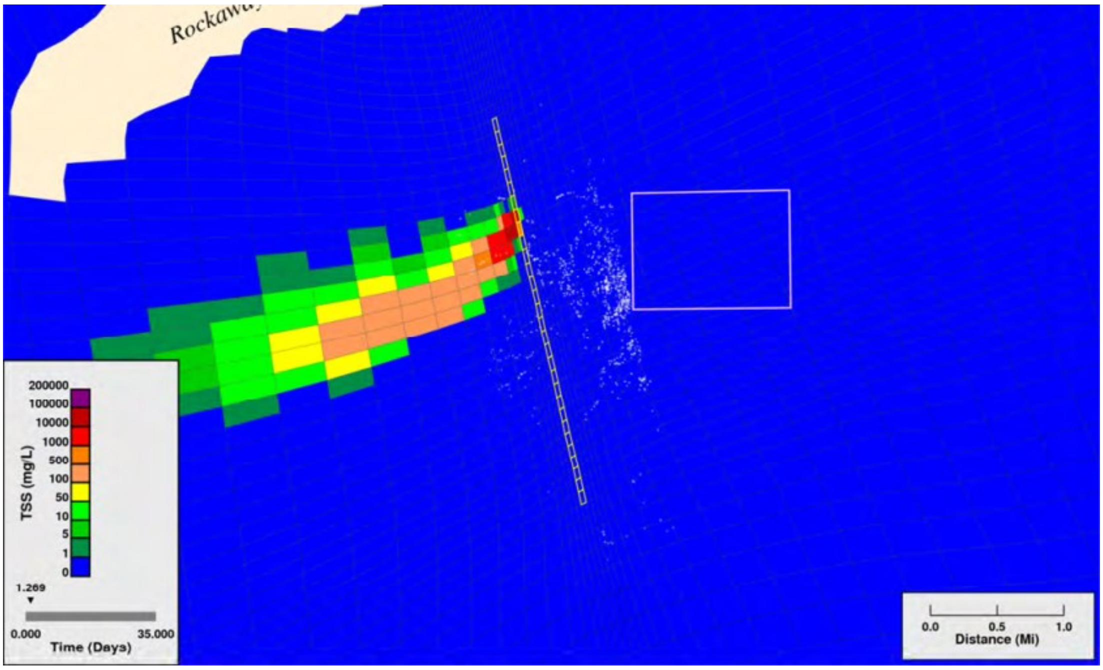


*Figure 5-3: Bottom layer projected solids concentration with rate of 366 m/hr, trenching 75% complete (Source: HDR, 2013)*

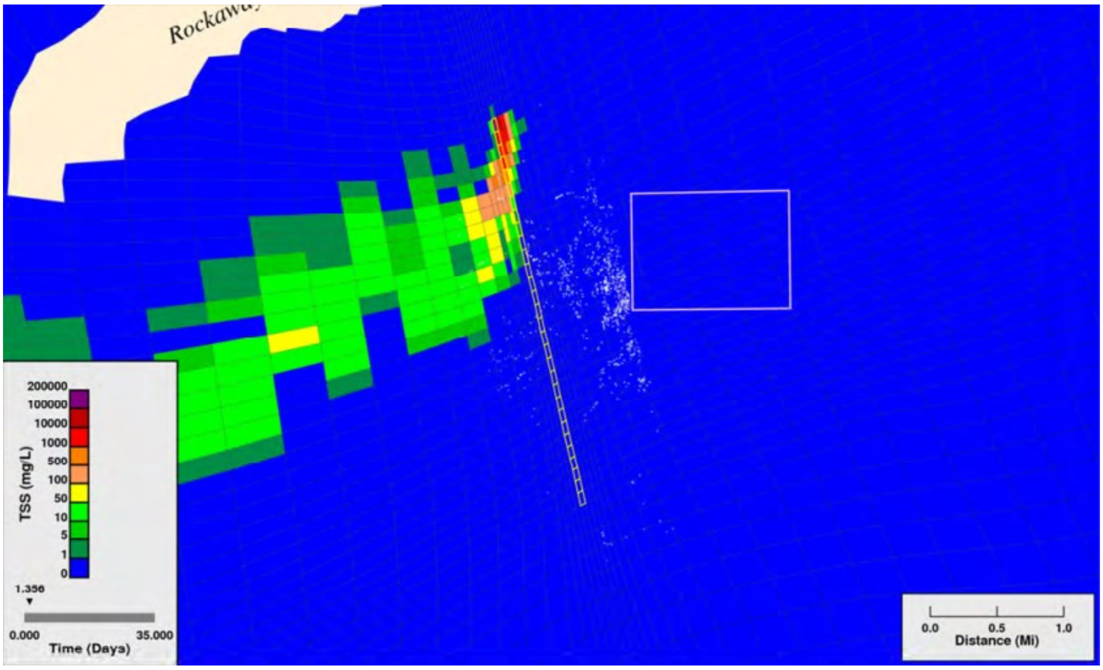


*Figure 5-4: Bottom layer projected solids concentration with rate of 366 m/hr, trenching 100% complete (Source: HDR, 2013)*

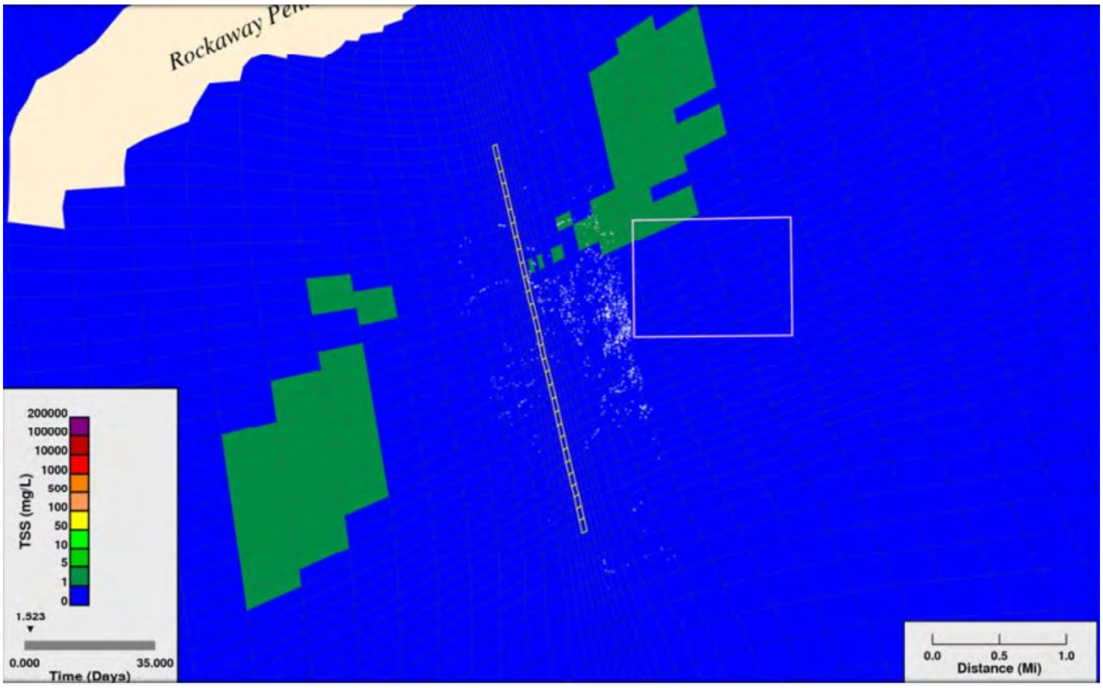


*Figure 5-5: Bottom layer projected solids concentration with rate of 366 m/hr, trenching 4 hours after completion (Source: HDR, 2013)*

This study found that the water column bottom layer solid concentrations exceeded 100,000 mg/l at the jet head and values of between 50 to 100 mg/L could occur at a distance of 4 km from the trench, although the water column plume dissipated within 4 hours following the end of construction. Solid deposition to the seabed greater than 0.3 cm occurred in a 630 m corridor adjacent to the trench. For slower production rates the water column plume exists for longer due to the longer construction duration, but the plume extent is smaller as the sediment mass released per unit time is smaller. Conversely for faster rates, the plume is present for shorter periods of time, but the extent is greater.

It is also evident that the majority of suspended sediment is within the lower part of the water column, with less significant disturbance higher up the water column (Figure 5-6).

#### 5.1.1.2 Assessment of effects of hydraulic jetting

Based on the range of production rates achieved in the USA case study (122 m/hr to 366 m/hr) each 800 m section of pipeline could be installed within 2.5 to 6.5 hours, with an average time of 4.5 hours. Making allowance for the new technology and tidal range, the construction process would need to be tested and optimised over a period of several weeks. Once the methodology was established, it is likely that each run of pipeline could be done over a period of several tides within a period of several weeks, with work initiating on a rising tide and completed on a falling tide. There would need to be localised disturbance around the connection between the land based and marine installed pipeline which would be done by barge mounted hydraulic excavators or from land once the majority of the pipelines were installed.

The disturbance will create significant rates of suspended sediment within the channel near the seabed at the location of the jet, with higher levels of suspended sediment that occur naturally at this location. The majority of the suspended sediment will be confined to the channel, with less than 50 to 100 mg/L expected on the intertidal area. These levels of suspended sediment are similar to observed suspended sediment concentrations on the intertidal areas.

The sediments will settle out within 1 to 2 tidal cycles and the net effect will be slight increases of the seabed which are unlikely to be measurable with standard hydrographic survey equipment.

#### 5.1.1.3 Assessment of effects of mass excavation

There are no directly applicable case studies for the modelling of mass excavation or data on rates of suspended sediment concentrations. However, based on our understanding of the physical processes, the sediment properties and the relatively shallow water depths we anticipate significant levels of suspended sediment concentration throughout the water column with this method. Based on this, we would expect sediment concentrations similar to those modelled for the hydraulic jet for the sea bed or 70% of the water depth as shown in d) and e) Figure 5-6. This is likely to result in a greater visible area of disturbance. However, the duration of high levels of concentration is still likely to be relatively short (4 to 6 hours) due to the sediment properties and localised to the location of the unit.

As this method is likely to occur during the higher parts of the tide due to the physical dimensions of the equipment, there is a greater risk of suspended sediment concentrations on the adjacent intertidal area with the potential of small amounts of fine sediments settling out. This is likely to be a short term effect as wind generated shear stress acting on the intertidal areas is likely to re-suspend these sediments and transport them to more acquiescent areas of the harbour.

This dispersive process is unlikely to result in any measurable change to the existing sedimentation patterns observed within the harbour.

If the excavated trench is required to be backfilled, this will also result in suspended sediment plumes during the deposition process. The extent of the plume will be a function of the sediment properties of the deposited material, but it is assumed that it will either be clean sands or gravels which are unlikely to result in any significant sediment plumes.

### 5.1.2 Short term effects of intertidal and transition area trenching

The intertidal dredging will be done by land based plant with dredged sediment removed to an approved offsite disposal area and the trench backfilled with granular backfill covered with a rock armour layer at the surface. A fixed silt fence will be installed around the seaward perimeter of the works area and the excavation is likely to be carried out at low tide to reduce interaction with the CMA. With these two measures in place there is unlikely to be any significant silt discharge outside the construction work area.

In the transition area works is proposed to be done by hydraulic back-hoe excavators mounted on a barge. The advantages of this approach include:

- The method minimises the amount of water added during the dredging process
- Ability to cope with a varied of materials including soft mud, firmer clays and weathered rock
- Good positional control
- Tried and proven in the Waitemata on other dredging projects.

There will be some suspended sediment released from the use of hydraulic excavators lifting sediment through the water column. Research over many dredge campaigns at the Ports of Auckland have indicated that the causes of sediment re-suspension using hydraulic excavators include:

- Disturbance of the seabed as the grab bucket enters and digs into the material (likely to be the main cause of sediment re-suspension)
- Spillage and wash-off of sediment as the bucket is lifted from the bed to the water surface

- Spillage from the bucket as it is swung over to the receiving hopper
- Overflows from the hopper
- Propeller wash and manoeuvring of vessels.

Japanese research indicates that as much as 80 kg/$m^3$ of sediment is released from large grab dredges. However, this is seen as a very conservative upper estimate of sediment release from a grab dredge that has different geometries to a backhoe dredge. However, assuming this mass is released and an hourly production rate of 200 $m^3$/hr, there is the potential to release 640 kg/day. At this location, due to the sediment properties of the seabed material which is predominantly sands with minor silt content (less than 10%), the released sediment will largely settle within 50 m of the dredged area. The remaining 10% will (i.e. 64 kg/day) remain in suspension longer, settling out over a period of 4 to 8 hours. It will travel in the direction of the tidal currents and be deposited on existing intertidal flat areas eventually to be re-suspended and deposited in more acquiescent parts of the harbour. It is anticipated that this effect is unlikely to increase suspended sediment rates on the water surface above 50 mg/l of ambient conditions at a distance of 100 m from the work area.

### 5.1.3 Te Wharau Creek

If the pipes are installed by HDD then there are no effects on coastal processes as the pipe is situated sufficiently below the level of the seabed.

## 5.2 Long term effects of marine trenching

Provided the pipes are installed at sufficient depth not to be exposed there should be no adverse long term effects of the structures on the physical coastal processes. The current plans of installing the pipes at least 1 m below the bed level mean that there is a minimum of 0.3 m of seabed cover over the pipes. The information on channel movement and bed level change is scarce, Figure 3-3 shows that up to 1.6 m of erosion occurred in the main channel in the vicinity of the pipe trench from 1854 to 1979. It is likely that this is more associated with channel migration rather than reduction in the base of the channel so the risk of further lowering exposing the pipes to bridging effects is low.

If the pipes are either placed shallower, or not backfilled, there are risks associated with under-scour and differential settlement that could put additional loads on the pipeline. This is more likely to occur where the pipeline is at an angle to the current, so in the transition areas from the main channel to the intertidal area rather than the pipes aligned within the main channel. However, this is more an effect on the pipeline with the effect to the environment expected to be less than minor, with a slight increase in sediment transport near the sea bed due to the scour process. There may be risks of anchor damage with an exposed pipe, but this is not expected to be significant due to the relatively small size of vessels being able to access this area.

Climate change effects are not expected to have any adverse effects of the proposed pipeline.

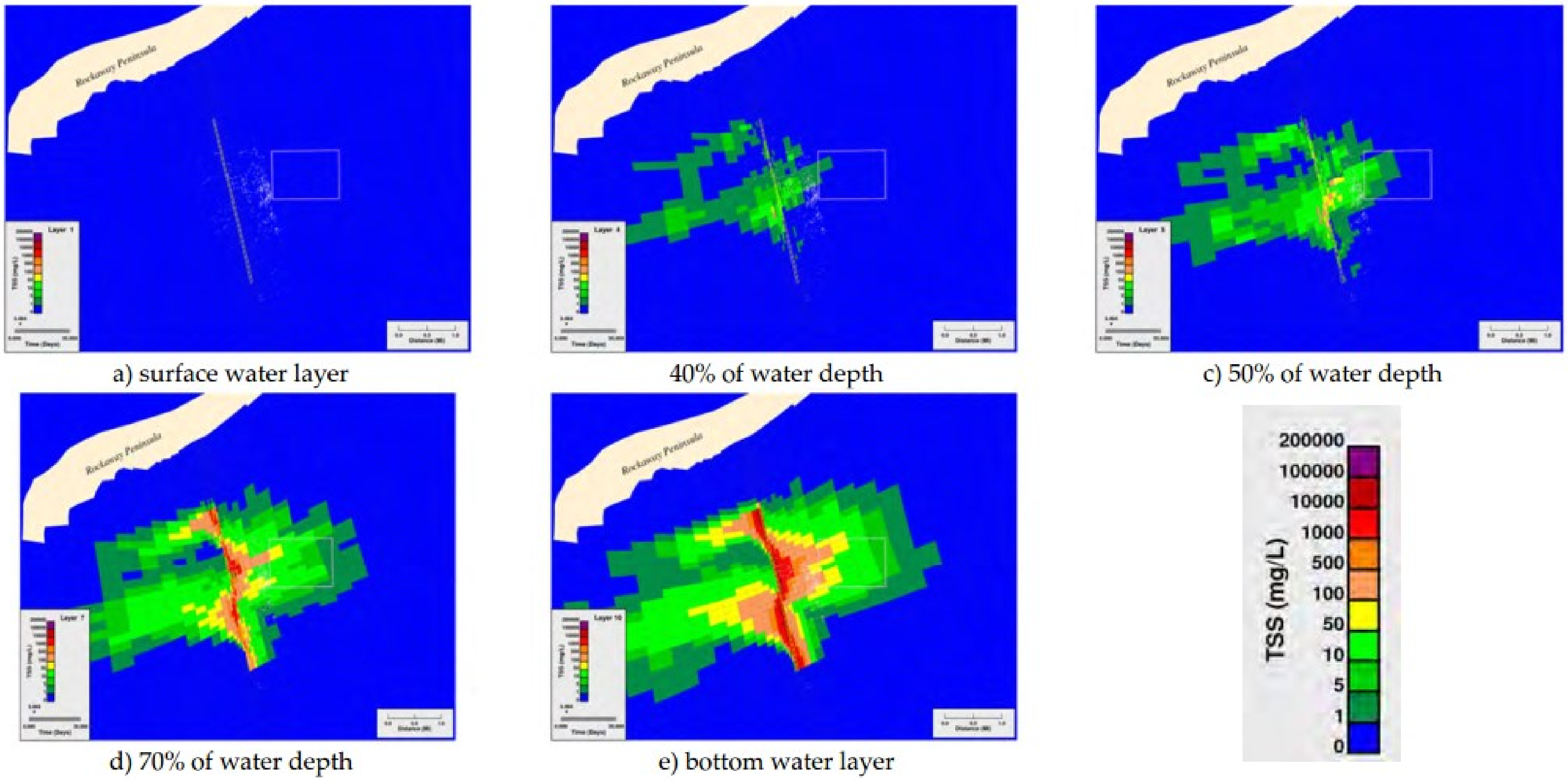


*Figure 5-6: Maximum simulated suspended sediment concentration in each cell within cell (Source: HDR, 2013)*

## 5.3 Summary

There should be no long term effect of the proposed works on the physical coastal environment with either HDD or marine trenching methods of installation.

The pipe trenching option comprising a combination of open trenching along the intertidal area and hydraulic jetting or mass flow excavation within the channel will have potential short term effects. Short term effects on the intertidal and transition area include localised increases in suspended sediment concentration. This is generally likely to be of a similar order of magnitude as the natural rates of suspended sediment along the intertidal area when wind generated waves exceed 0.2 m in height.

The hydraulic jetting/mass flow evacuation in the subtidal channel and connecting the intertidal and subtidal pipelines will have a short term (less than 1 to 2 tidal cycles) effect of significant increases in suspended sediment at the location of the jetting head during the periods of activity, and this will largely be confined to the tidal channels. The mass flow evacuation option is likely to result in a greater spatial extent of elevated suspended sediment concentrations compared to the hydraulic jetting with the potential for greater short term deposition on the adjacent intertidal area, but still only occurring over a short period of time.

If the pipes are installed by HDD then there are no effects on coastal processes as the pipe is situated sufficiently below the level of the seabed

# 6 Recommended mitigation and management measures

The main option to avoid potential effects on the coastal environment would be to complete the pipe installation through HDD techniques under the harbour.

However, the proposed trenching methods for the intertidal and the interface works to connect the intertidal and subtidal works are relatively small scale and localised. If required, the extent of disturbance could be confined with the installation of a silt fence. The silt fence could be fixed on the seabed for the intertidal work, but would need to be sufficiently robust to withstand localised wind generated waves. For the interfacing connection works would be carried out at high tides when tidal currents were low to avoid the requirements of a floating silt curtain.

For the subtidal hydraulic jetting/mass flow excavation will create greater levels of suspended sediment migration. However, these high sediment loads are of a short duration and are localised to the vicinity of the jetting head and its surrounds.

A mobile floating silt curtain is not a viable method to reduce the potential visible silt plume for the upper part of the water column. A fixed floating silt curtain around the entire perimeter of the works is also unlikely to be viable as it would create significant obstruction to other users in the harbour and is not considered practicable.

The most effective method for reducing the extent of the suspended sediment concentration could be reduced by limiting work to periods of slack high tide (i.e. 2 hours either side of high or low water if the size of plant allows) where tidal currents are low. This would require a longer construction period and a greater duration of disturbance. It is anticipated that the most effective method would be established during trials to optimise performance prior to the installation works progressing.

# 7 Applicability

This report has been prepared for the benefit of Watercare Services Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

Tonkin & Taylor Ltd

Report prepared by:

..........................................................

Richard Reinen-Hamill

Senior Coastal Engineer

Authorised for Tonkin & Taylor Ltd by:

..........................................................

Peter Roan

Project Director

RRH
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# Appendix A: Bibliography/references

Beca (2009) Coastal processes in the Upper Waitemata Harbour. Report prepared for Hobsonville Land Company. Beca Infrastructure Ltd, August 2009.

Bell, R.G. (2013) Submission on the Proposed Northland Regional Policy Statement of Evidence, 26 May 2013. 22p.

Bell, R. (2012) Sea Levels for New Zealand: Give us a Number Please! *Presentation at Sea level Rise: meeting the Challenge Conference*, Wellington May 10-11, 2012.

Dolphin, T.J. and M.O. Green (1997) Sediment dynamics of an estuarine "Turbid Fringe". Combined Australasian Coastal Engineering and Ports Conference, Christchurch September 1997

HDL/HydroQual (2013) Hydrodynamic and sediment transport analysis for Rockaway delivery lateral project. Unpublished report prepared for Ecology and Environment, Inc. New York, 30 April 2013.

Hume, T. (1983). Upper Waitemata Harbour sediments and inferred impact of future catchment and estuary use change. Water Quality Centre Report No. 3, Hamilton, 1983

IPCC (2013). Working Group I contribution to the IPCC 5th Assessment Report "Climate Change 2013: The Physical Science Basis". DRAFT report by Intergovernmental Panel on Climate Change. June, 2013

Kober, J.H., F.D. Messina, D. Dean (2001) Advances in jet assisted plowing. Underwater Magazine, July/August 2001.

McLachlan, M.J. and T.M. Hume (1981). Data file – Lucas Creek tidal survey, 11 March 1981, Upper Waitemata Harbour Catchment Study, Hamilton Science Centre Internal Report No. 80/6

NIWA Ecosystems (1997). Effects of urbanisation in the catchment of the Upper Waitemata Harbour/ three reports prepared by NIWA Ecosystems for Auckland Regional Council. Auckland, New Zealand: Auckland Regional Council

NIWA (2004a). Prediction of contaminant accumulation in the Upper Waitemata Harbour. Niwa report ref HAM2003-087/1 report prepared for Auckland Regional Council, North Shore City, Rodney District Council, Waitakere City Council and Transit New Zealand, June 2004.

NIWA (2004b). Water quality survey of Mahurangi Harbour, Upper Waitemata Harbour and Tamaki Estuary Annual Report. Niwa report ref HAM2004-045 report prepared for Auckland Regional Council, October 2004.

NIWA (2013) Coastal inundation by storm tides and wave in the Auckland region. NIWA report ref. HAM2013-059 prepared for Auckland Council, September 2013.

Oldman, J, N. Hancock, M. Lewis (2008) Central Waitemata Harbour Contaminant Study. Hydrodynamics and Sediment Transport Fieldwork. Prepared by NIWA for Auckland Regional Council. Technical Report 2008/036

Swales, A., Hume, T. M., McGlone, M. S., Pilvio, R., Ovenden, R., Zviguina, N., Hatton, S., Nicholls, P., Budd, R., Hewitt, J., Pickmere, S., & Costley, K. (2002). Evidence for the physical effects of catchment sediment runoff preserved in estuarine sediments: Phase II (field study).

Williams B.L. and J.C. Rutherford (1983). The flushing of pollutants and nutrients from the Upper Waitemata Harbour. Upper Waitemata Harbour Catchment Specialist report, Auckland Regional Authority, Auckland.

# Appendix B: Hydrodynamic Model Study

# REPORT

## Greenhithe Bridge Watermain Duplication and Northern Interceptor

### Coastal Modelling Report

**Prepared for:**

**Watercare Services Ltd**

**March 2015**

Job No: 29719.v2

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

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# 1 Introduction

A computational hydrodynamic model was developed of the middle and upper portions of the Waitemata Harbour to assist in the assessment of effects on the physical coastal processes operating within the harbour area between Hobsonville and Greenthithe. The model was used the calibrated model used to establish effects of the Waterview Connection Project extended to include the upper harbour area.

This computational model suite is an industry accepted model for simulating coastal and inland flows. It has previously been successfully applied to several coastal areas of New Zealand, including the Waitemata Harbour.

# 2 Numerical model

The hydrodynamic module of MIKE 21 FM solves the vertically integrated equations for fluid flow (conservation of continuity and momentum, i.e., the Saint Venant equations) over a grid consisting of non-uniformly sized triangles and quadrilaterals. This 'flexible mesh' allows for maximum computational efficiency, while maintaining good resolution in areas of interest, such as the Upper Harbour Bridge and surrounding area of interest.

## 2.1 Model domain and resolution

The hydrodynamic model provides spatially and time varying information of water levels and current velocities, when provided with the bathymetry and the time-varying forcing (tides, currents) along the model boundaries. For this assessment, the inputs to the model were tidal water levels (with horizontal velocity) at the boundary of the computational domain, and sources of mass and momentum representing flow into harbour.

The computational model covers the area of the Waitemata Harbour, from the Auckland Harbour Bridge in the east to the Rangitouni Stream in the north, and includes the Whau River, Waterview Estuary and Oakley Inlet to the south as shown in Figure 2-1.

Mesh size has been refined around the proposed causeway with grid size of approximately 2 $m^2$ – 5 $m^2$ at the coastal boundary, increasing to approximately 20 $m^2$ at the Lukas Creek mouth and 50 $m^2$ further west (as shown in Figure 2-2 and Figure 2-3). Mesh resolution south of the Upper Harbour Bridge (old model domain) has not been altered.

Due to the mesh generation processes, results within 2 -3 grid size cells of the proposed SH18 causeway widening should be treated with caution and compared with results of the adjacent cells for consistency.

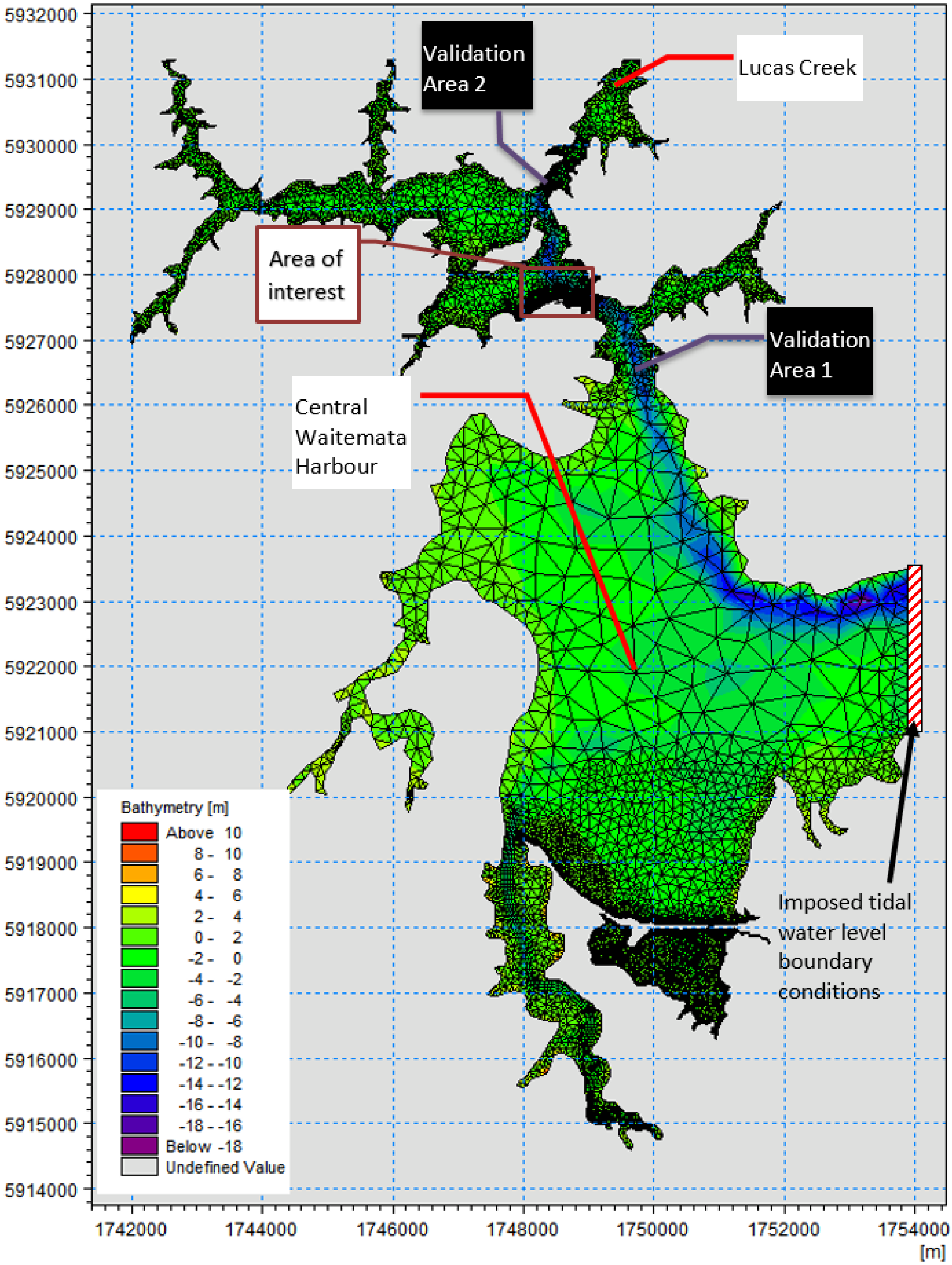


*Figure 2-1: Model domain, boundaries and validation points*

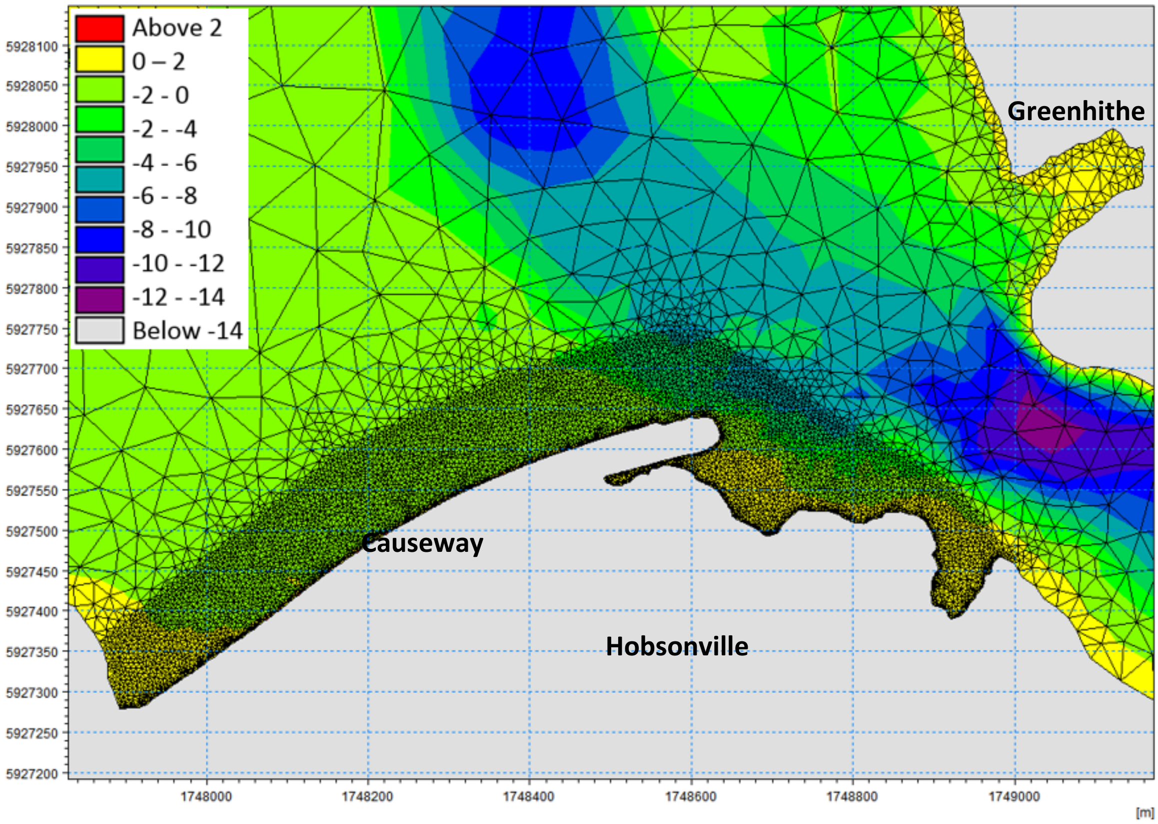


*Figure 2-2: Flexible mesh grid configuration at area of interest for existing bathymetry*

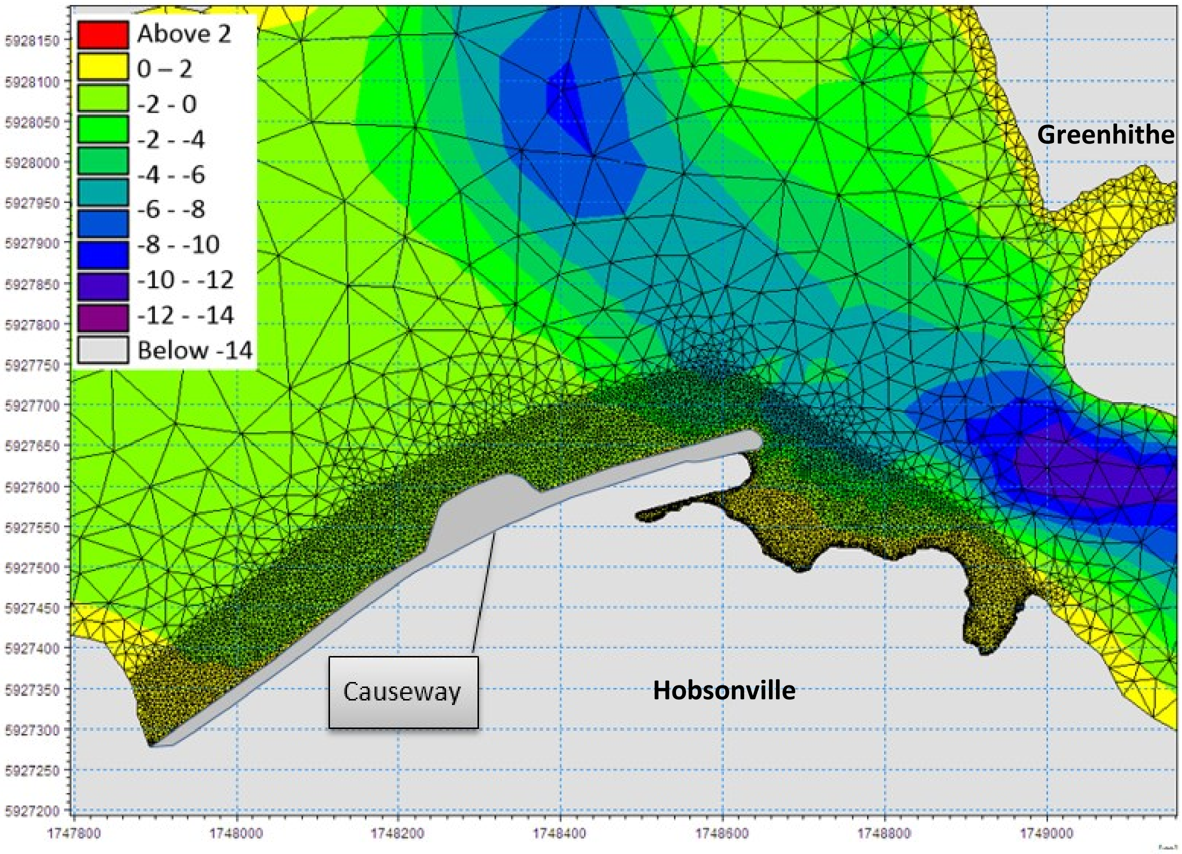


*Figure 2-3: Flexible mesh grid configuration at area of interest for proposed bathymetry*

## 2.2 Bathymetric data

### 2.2.1 Sources

The model bathymetry was obtained from several sources, these are:

- The existing MIKE 21 'Regional Harbour Model' (RHM) of the complete Waitemata Harbour and inner Hauraki Gulf
- Boat surveys provided by NIWA and Marine Discovery Ltd (in the Waterview Estuary, Oakley Inlet and Whau River areas)
- Causeway drawing for existing and proposed scenarios, provided by Watercare Services Ltd
- Bathymetric Survey provided by Scantec Geophysical Consultants (for Upper Harbour Bridge area)
- LiDAR (provided by Auckland Local Government Geospatial information), in the upper harbour region and aerial photographs
- LINZ depth contours.

These data sources were combined to generate the bathymetric grid shown in Figure 2-1.

### 2.2.2 Hierarchy

Data sources for the original Waterview model is described in detail in the Tonkin & Taylor, NIWA, 2010 Waterview model report. The table below provides more detailed description of the additional data used to assess effects of the proposed SH18 causeway widening

| Layer number | Layer name | Comment | Exceptions |
|---|---|---|---|
| 1 | MIKE21 RHM | Used in all areas of the original Waterview model (excluding areas of higher resolution data) | |
| 2 | Boat surveys (NIWA and Marine Discovery Ltd) | Used in the Waterview Estuary, Oakley Inlet and Whau River areas | |
| 3 | Causeway drawing for existing and proposed scenarios, (WaterCare) | Used to reflect levels adjacent to the coast line for the existing and proposed scenarios | Around the bridge piers the existing depth has not been used due to it reflecting the level of the bridge, in this area the intertidal LiDAR data set is found to be more appropriate and is used |
| 4 | Bathymetric survey (Scantec Geophysical Consultants) | Used in the vicinity of the Upper Harbour Bridge area and southern bridge approach | In areas where there is an overlap between the Causeway drawings (in 3) and the bathymetric survey, the Causeway drawings data set has been used |
| 5 | Intertidal Lidar | Used in the Upper Harbour Bridge area (e.g. around piers and areas north of the upper harbour bride | |
| 6 | Depth contour (LINZ website) | Used at locations with no other data sets. i.e. not intertidal LiDAR nor any bathymetric survey data, e.g. Lucas Creek | |

### 2.2.3 Datum and projection

The datum utilised in the model is Auckland Vertical Datum 1946 (AVD46). The projection is in New Zealand Transverse Mercator (NZTM).

## 2.3 Boundary conditions

The boundary conditions used in the model correspond to a typical spring tide and neap tide (Figure 2-4). All simulations used a nominal start date, as opposed to an actual period. Water flow in the model is driven by a time series of tidal water levels at the eastern boundary of the model. This time series was obtained from the calibrated RHM for both spring and neap tides.

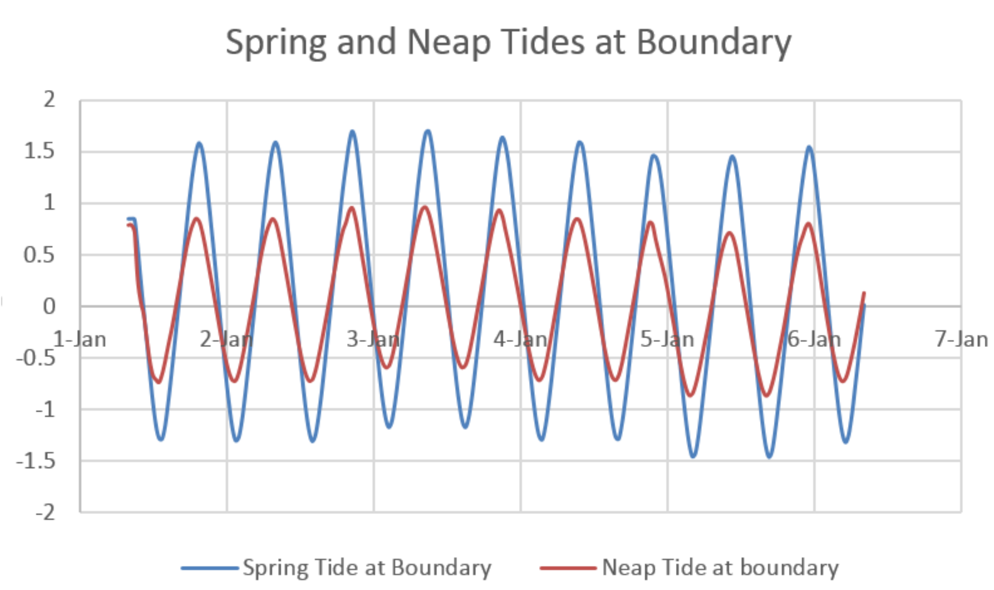


*Figure 2-4: Spring and neap tides at model boundary*

## 2.4 Model validation

The results of the model were verified using the 2010 Waterview model data set.

Surface elevation data near the 2010 northern boundary in the 2014 model (south of the Upper Harbour Bridge), was compared with the 2010 numerical model boundary condition used for the Waterview project.

A difference plot (Figure 2-5) provides a comparison between the time series. The RMSE for this data is 0.06 m showing a reasonable match between the two datasets.

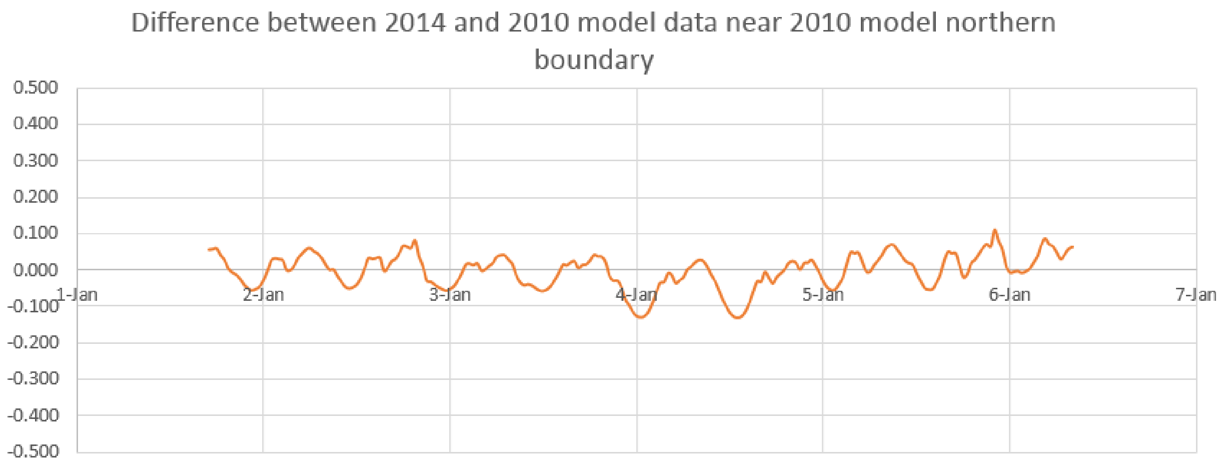


*Figure 2-5: Validation - surface elevation showing the difference between the 2014 and 2010 model.*

The original Waterview model validation can be found in Tonkin & Taylor, NIWA 2010 Waterview model report.

# 3 Results and analysis

## 3.1 Scenarios

Four scenarios were simulated, these are:

1. Existing bathymetry with spring tide
2. Proposed bathymetry ( including causeway widening and extension) with spring tide
3. Existing bathymetry with neap tide
4. Proposed bathymetry (including causeway widening and extension) with neap tide.

Sections 3.2 to 3.4 of this report describe the results, including spatial plots for surface elevation, flood and ebb current speed, and flood and ebb shear stress for the spring tide.

In addition to the spatial plots provided in the sections above, data sets in specific locations as shown in Figure 3-1 have also been obtained. These are included in Appendices A and B. Data sets in appendices include:

- Tables showing maximum current speed at flood and ebb tides as well as corresponding depth and surface elevation for spring and neap tides (Appendix A)
- Time series showing surface elevation, current speed and total water depth for spring and neap tides (Appendix B).

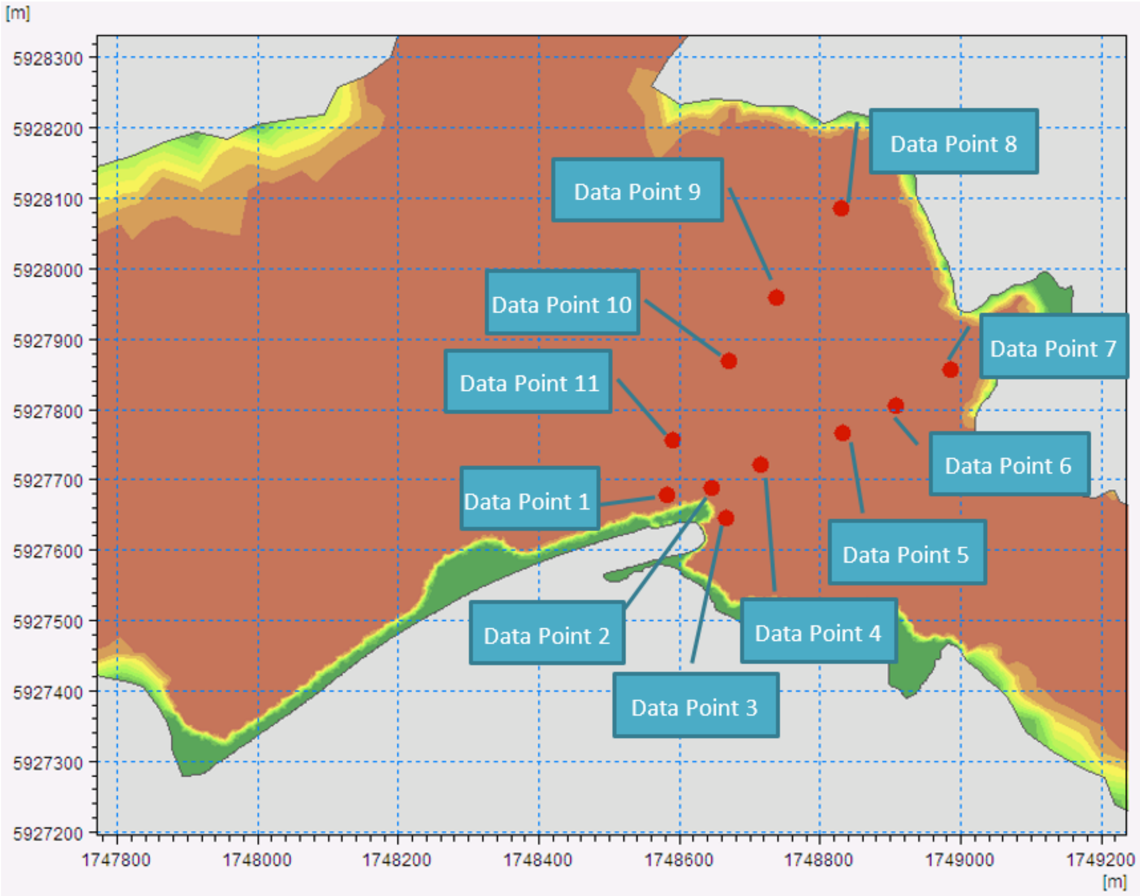


*Figure 3-1: Location of output points for comparison of changes*

## 3.2 Surface elevation

Model results include surface elevation (water level). For the existing scenario, in the vicinity of the Upper Harbour Bridge, the spring tide maximum surface elevation (shown in Figure 3-2) is 1.76 mRL to 1.78 mRL.

For the proposed scenario, in the vicinity of the Upper Harbour Bridge, the spring tide maximum surface elevation (shown in Figure 3-3) is also 1.76 mRL to 1.78 mRL.

Existing and proposed spring tide maximum surface elevation was compared through a difference plot. The maximum difference in water level (proposed scenario - existing scenario) is approximately - 0.0015 m to 0.001 m as is shown in Figure 3-4. This very small difference in levels suggests negligible changes to surface water levels as a result of the causeway widening and extension and these changes are unlikely to be measurable.

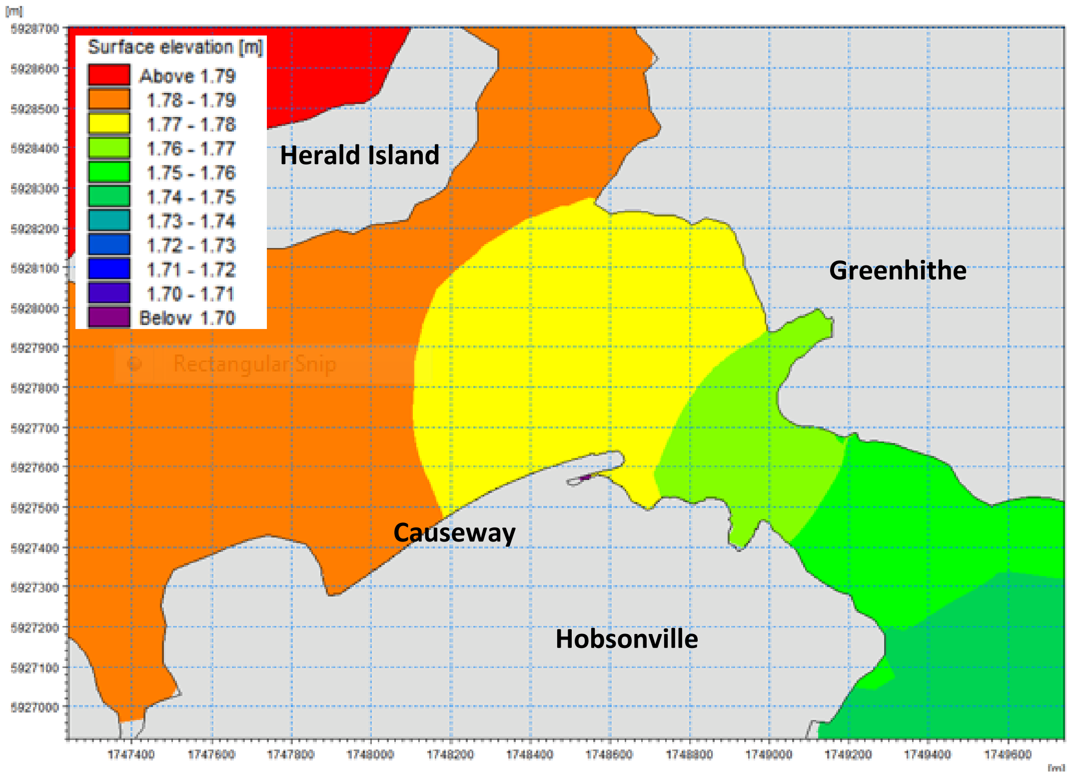


*Figure 3-2: Existing scenario spring tide maximum surface elevation (Scenario 1)*

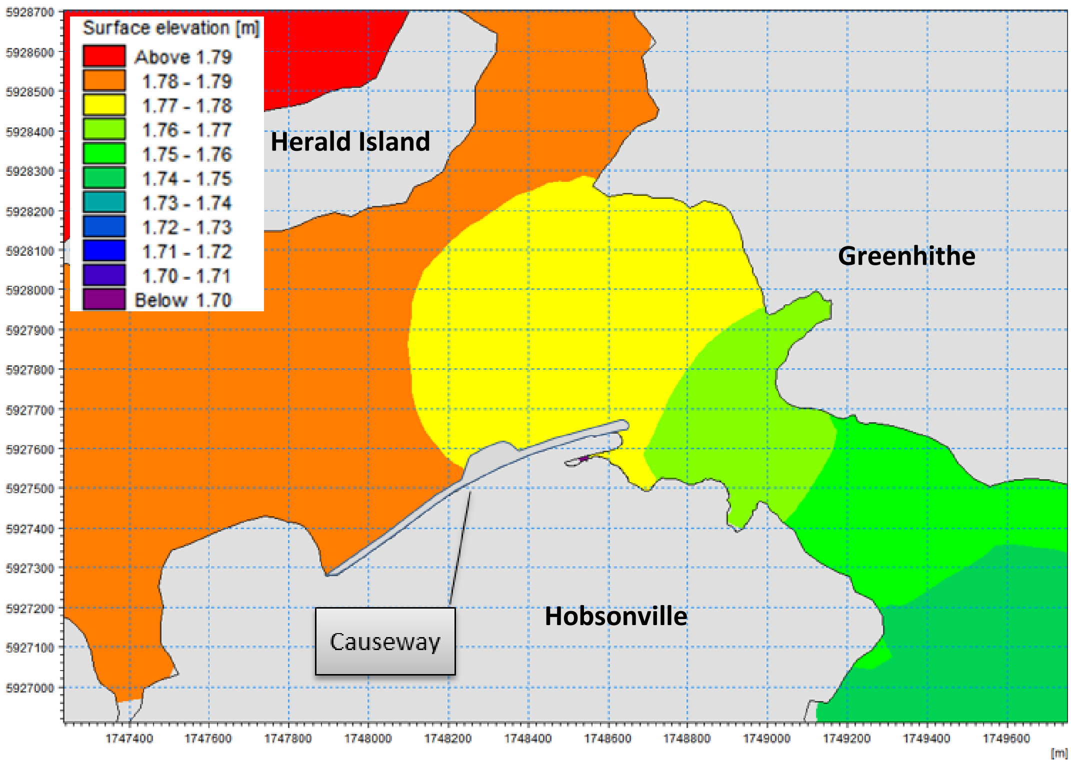


*Figure 3-3: Proposed scenario spring tide maximum surface elevation (Scenario 2)*

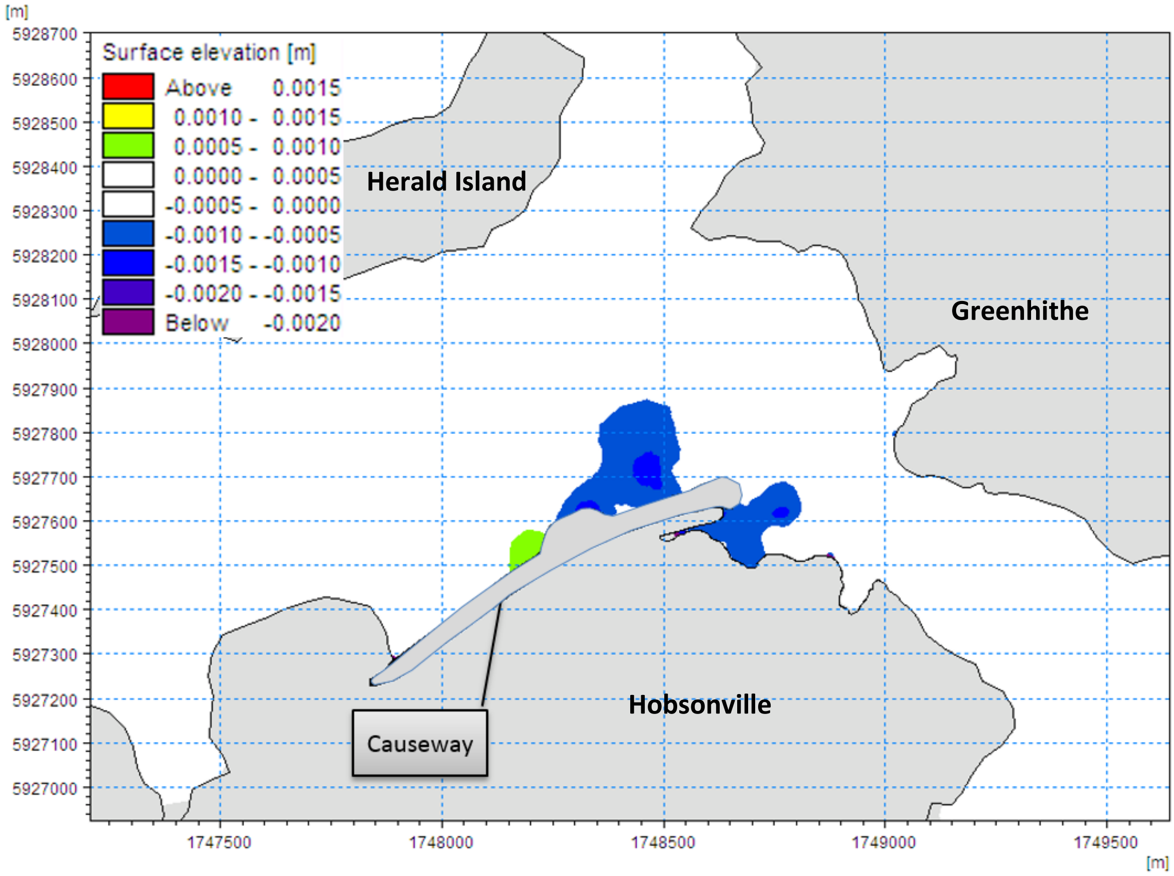


*Figure 3-4: Difference between Scenario 1 and 2 in spring tide maximum surface elevation*

## 3.3 Peak current speed

Model results also include peak current speed information (depth averaged velocity). Current speed was extracted and compared between existing and proposed scenarios, for typical flood and ebb tides.

### 3.3.1 Current speed – flood tide

In the vicinity of the Upper Harbour Bridge, the peak spring flood tide current speed ranges between:

- 0.1 m/s and 0.8 m/s for the existing scenario (shown in Figure 3-5) and the proposed scenario (shown in Figure 3-6).

Although this is essentially the same for both scenarios, minor differences in current speed are observed in several locations. This is shown in Figure 3-7, where:

- An increase in current speed is shown in the model results near the tip of the proposed causeway and is up to 0.1 m/s
- A decrease in current speed is shown in the model results immediately north and south of proposed causeway tip and is up to -0.25 m/s.

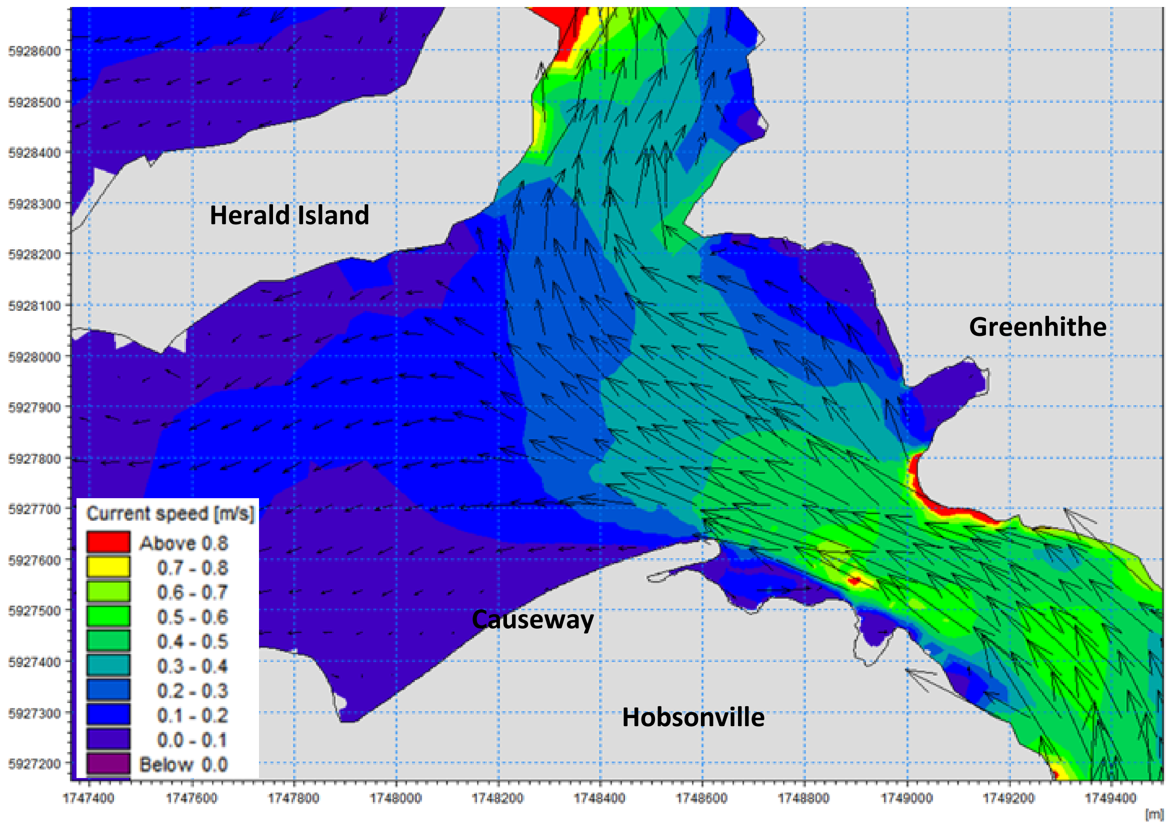


*Figure 3-5: Existing situation (Scenario 1) - spring tide maximum flood current speed*

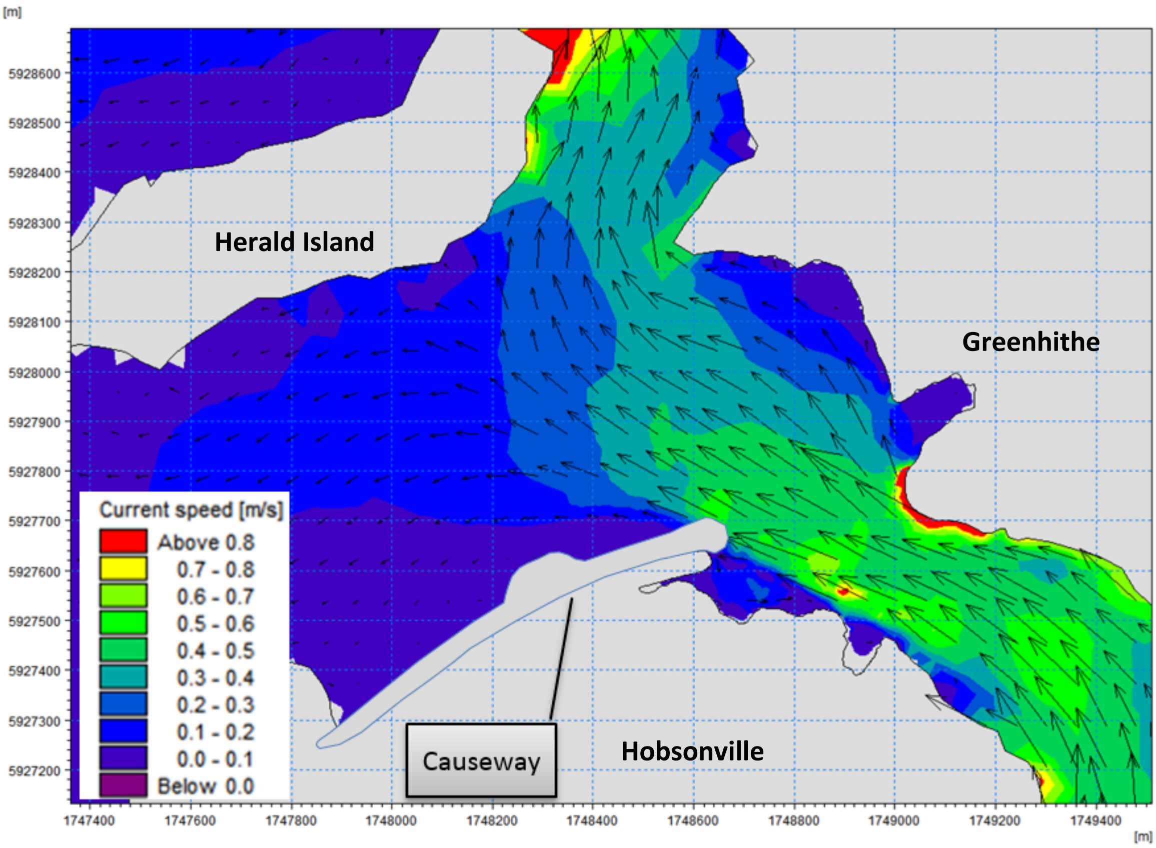


*Figure 3-6: Proposed bathymetry (Scenario 2) - spring tide maximum flood current speed*

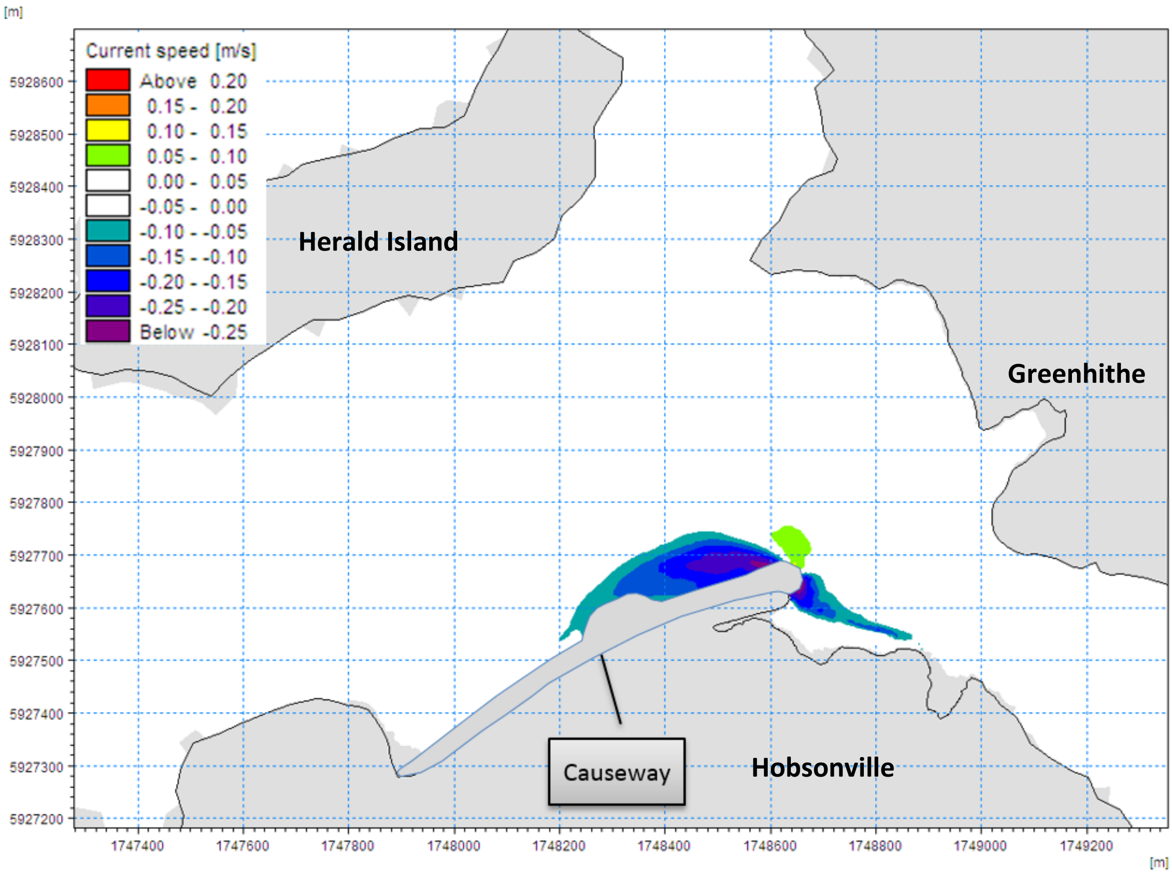


*Figure 3-7: Difference in flood current speed for spring tide*

### 3.3.2 Current speed – ebb tide

In the vicinity of the Upper Harbour Bridge, the peak spring ebb tide current speed ranges between:

- 0.1 m/s and 0.8 m/s for both Scenario 3 (shown in Figure 3-8) and Scenario 4 (shown in Figure 3-9).

Although this is essentially the same for both scenarios, minor differences in current speed are observed several locations. This is shown in Figure 3-10, where:

- An increase in current speed between is shown in the model results near the tip of the proposed causeway and is up to 0.1 m/s
- A decrease in current speed is shown in the model results immediately north and south of proposed causeway tip and is up to -0.25 m/s.

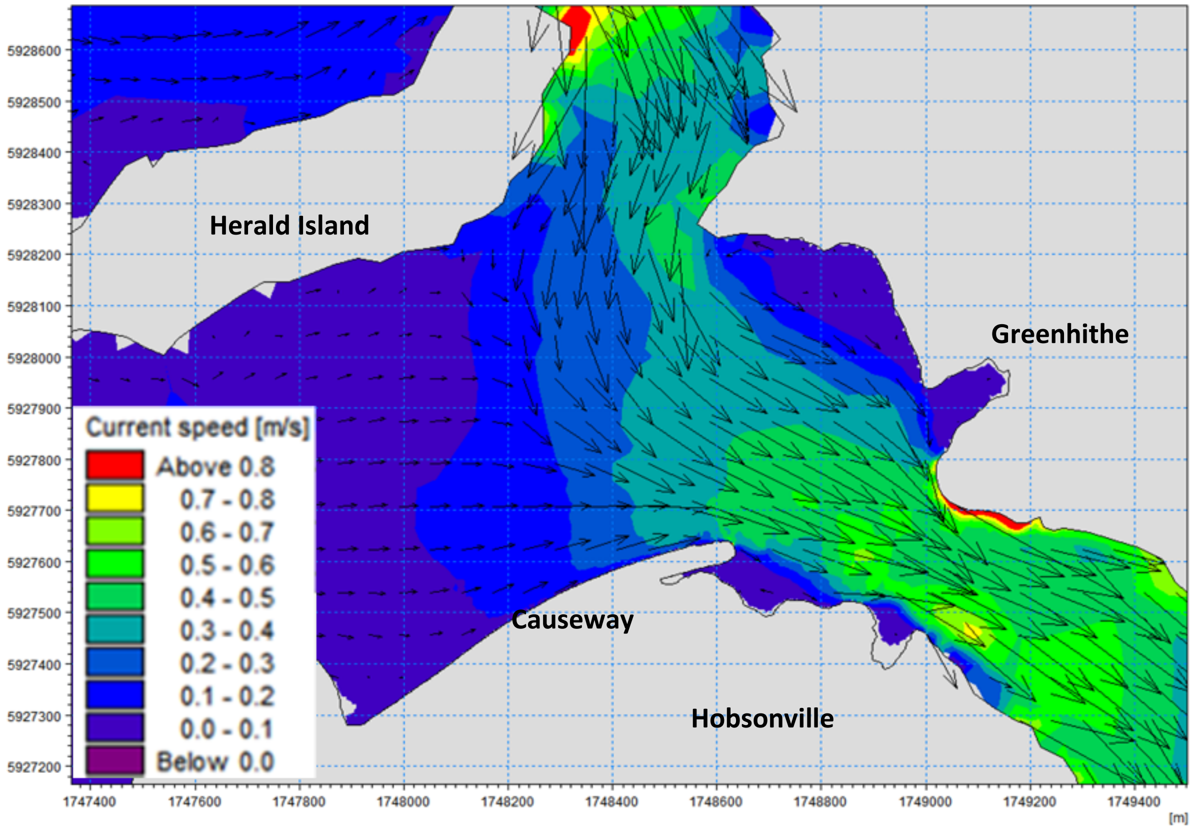


*Figure 3-8: Existing bathymetry (Scenario 1) - spring tide maximum ebb current speed*

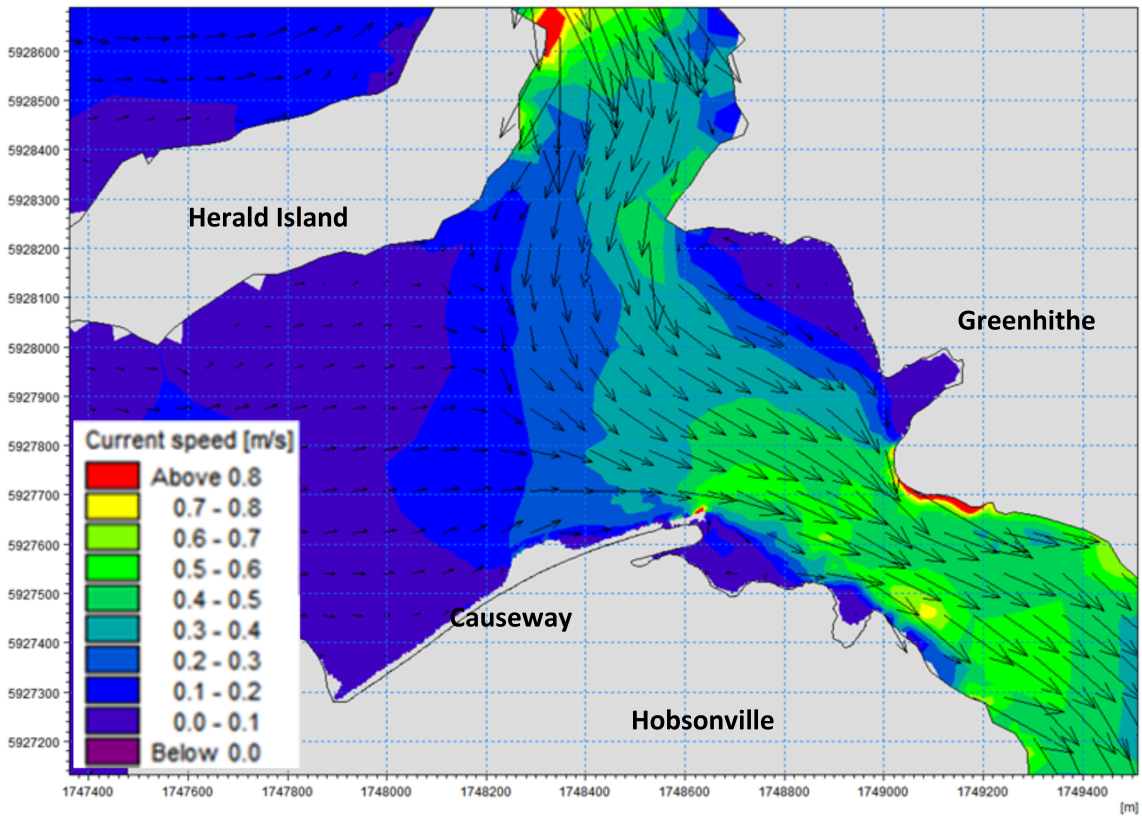


*Figure 3-9: Proposed bathymetry (Scenario 2) - spring tide maximum ebb current speed*

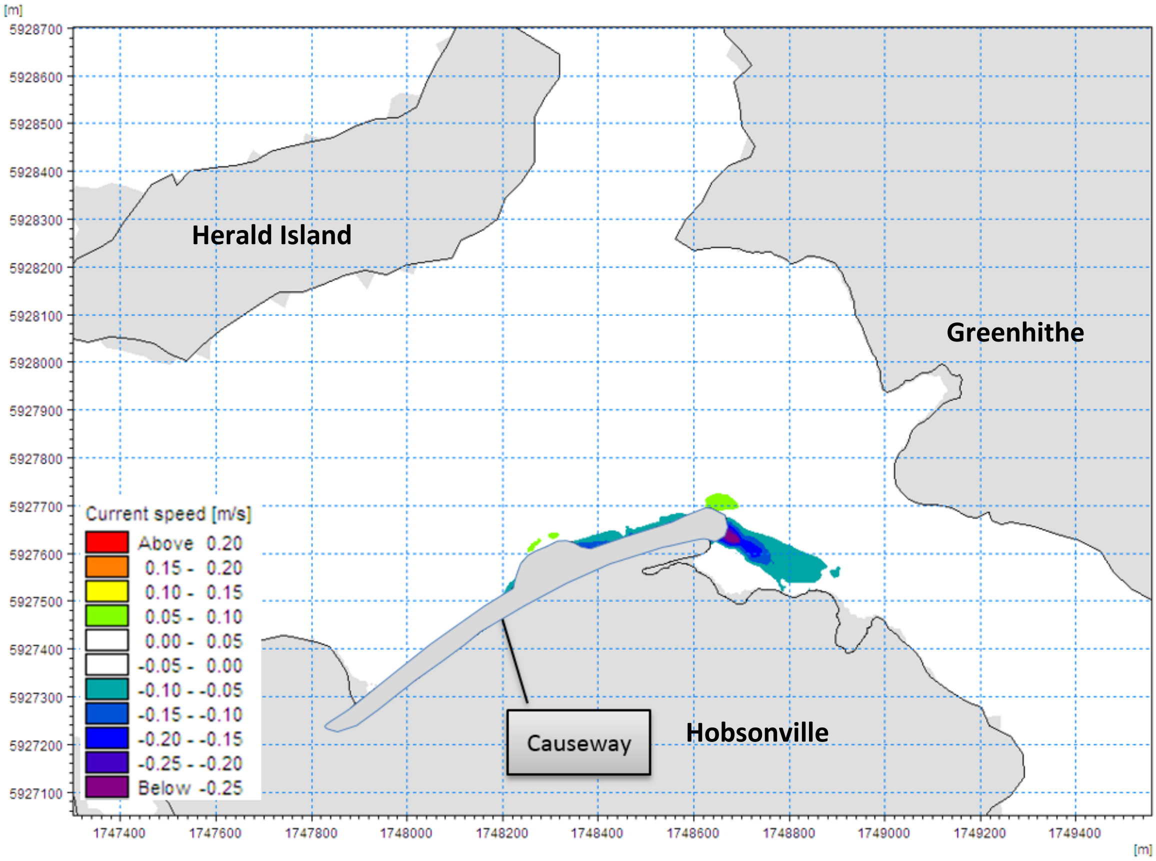


*Figure 3-10: Difference in ebb current speed for spring tide*

Current speed information at two specific locations for both spring and neap tides are presented below. This is to further illustrate the effects of the proposed causeway on current speed in the vicinity of the bridge. The two data points chosen for this discussion are:

- Data point 2: this is chosen as it represents an area affected by the proposed causeway
- Data point 5: this is chosen as it is considered representative of the channel under the bridge.

Time series and peak current speed information from all data points are included in Appendix A and B.

**Table 3.1: Maximum current speed for spring and neap tides during flood and ebb tides**

| Location (refer Figure 3-1) | maximum current speed (m/s) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Flood existing | Flood proposed | Difference in flood tide (m/s, %) | | Ebb existing | Ebb proposed | Difference in ebb tide (m/s, %) | |
| Data Point 2 spring tide | 0.478 | 0.565 | 0.087 | 18.2% | 0.462 | 0.548 | 0.086 | 18.6% |
| Data Point 2 neap tide | 0.234 | 0.272 | 0.038 | 16.2% | 0.245 | 0.285 | 0.041 | 16.7% |
| Data Point 5 spring tide | 0.497 | 0.504 | 0.007 | 1.4% | 0.464 | 0.48 | 0.016 | 3.4% |
| Data Point 5 neap tide | 0.244 | 0.247 | 0.003 | 1.2% | 0.251 | 0.26 | 0.01 | 4.0% |

Table 3.1 shows that:

- The differences in current speed between the proposed and existing scenarios for a typical flood tide at representative locations under the bridge and in the vicinity of the causeway are:
  - Increases of up to 0.087 m/s near the proposed cause way and 0.007 m/s under the Upper Harbour Bridge for the spring tide
  - 0.038 m/s near the proposed cause way and 0.003 m/s under the Upper Harbour Bridge for the neap tide.
- The differences in current speed between the proposed and existing scenarios for a typical ebb tide at representative locations under the bridge and in the vicinity of the causeway are:
  - 0.086 m/s near the proposed cause way and 0.016 m/s under the Upper Harbour Bridge for the spring tide
  - 0.041 m/s near the proposed cause way and 0.01 m/s under the Upper Harbour Bridge for the neap tide.

As expected at this location peak current speed is higher during spring tide than during neap tide for both flood and ebb tides. The difference between the existing and proposed scenario is also larger during neap tides than flood tides. Changes at Data Point 5 are negligible (less than 4%). With physical measurement accuracy limited to around ±0.008 m/s the modelled changes under the Upper Harbour Bridge are unlikely to be measurable.

Changes in velocity at other data points are shown in Table A1-3 and A1-6 (Appendix A) for spring and neap tides respectively. Table A1-3 table shows the reduction in velocity at Data Points 1 and 3 and negligible changes at Data Points 5, 6, 7 and 8. Within the main channel there are slightly greater velocities during flood tide conditions (Data Points 9, 10 and 11) and a slight increase at Data Point 8 on the edge of the main channel. The remaining points show negligible change (less than 4%). This confirms the effects of the causeway extension are localised around the end of the extension as shown in Figure 3-7 and Figure 3-10. During neap tide conditions, velocities are lower but the percentage changes are in a similar order (Table A1-6). The only difference in trend can be seen at Data Point 11 where neap ebb tide conditions change to a very slight reduction in velocity compared to no change during spring tide.

Overall, velocity changes are not likely to significantly affect sediment transport regimes. In the main channel existing velocities are able to transport sediment, while on the intertidal areas velocities are generally lower than the threshold to move sediment and the proposed changes do not change the velocity regimes. However, this is examined in more detail in the following section which looks at erosion and accretion potential via an analysis of shear stress.

## 3.4 Shear stress

The shear stress at the sediment-water interface generated by water flowing over the bed surface is a primary determinant of the extent to which materials settling out of the water column are deposited on the bed or are eroded from it, and whether particles in motion are transported as suspended load or bed load. Given maximum the flood and ebb current speed (and corresponding water depth), peak shear stress due to tidal currents can be calculated using formula 2.2.17 from Leo C. van Rijn (1993):

$$\tau_{b,c} = \rho \, \mathrm{g} \, \mathrm{h} \, \mathrm{I} = \rho \, g \, \frac{\overline{\mathrm{u}}^2}{\mathrm{C}^2} = \frac{1}{8} \rho \, \mathrm{f}_c \, \overline{\mathrm{u}}^2$$

Where:

$\mathrm{h} =$ water depth (m)

$\mathrm{I} =$ energy line gradient (-)

$\overline{\mathrm{u}}$ = depth-averaged peak velocity (m/s)

C = Chezy-coefficient ($C^2 = 8g/f_c$), ($m^{0.5}/s$)

$\mathrm{f}_c =$ Fraction factor of Darcy Weisbach (-) $= \mathrm{f}_c = 0.24 \left[\log\left(\frac{12h}{k_s}\right)\right]^{-2}$

$k_s$ = effective bed roughness height (m), for a plane bed $k_s = 3d_{90}$*1

$\rho$ = fluid density ($kg/m^3$) = 1024

$g =$ acceleration of gravity ($m/s^2$)

*1 Assuming $d_{90} =$ 0.4 mm based on PSD for 'Upper Waitemata Harbour Borehole 3, Sample 3, Depth 0.8 m. This is a conservative close to lower bound based on sediment sampling where values ranges from 0.25 mm to 40 mm and the average value was 2 mm.

The resulting shear stress plots are shown in Figure 3-11 to Figure 3-16.

Erosion occurs when grain shear stress at the sediment surface exceeds the critical shear stress. This can be estimated from the formula of Guo (2002):

$$\tau_{ce} = (G_p - 1)\rho g d_p \left(\frac{0.23}{d_*} + 0.054\left[1 - exp\left(-\frac{d_*^{0.85}}{23}\right)\right]\right)$$

And

$$d_* = d_p \left[\frac{(G_p - 1)g}{\vartheta^2}\right]^{-1/3}$$

Where:

τce = critical shear stress for erosion [ML-1T-2]

Gp = sediment particle specific gravity ≈ 2.65 [dimensionless]

dp = sediment particle diameter

d* = dimensionless particle diameter [dimensionless]

v = kinematic viscosity [L2T-1]

Using this formula the critical bed-shear stress for the lower bound of sediment grading is around 0.2 N/m2 and is around 0.8 N/m2 for average grain size properties. This means that where shear stress values are less than 0.2 N/m2 no erosion due to tidal currents are expected and between 0.2 N/m2 and 0.8 N/m2 some erosion of finer sediments can be expected.

### 3.4.1 Shear stress – flood tide

The shear stress for the existing situation is shown in Figure 3-11. Figure 3-12 shows the shear stress for the proposed development and Figure 3-13 shows the difference in shear stress. The intertidal areas all have shear stress values of below 0.2 N/$m^2$ indicating no erosion due to tidal currents are likely in these areas. Within the tidal channel shear stress values range from 0.2 N/$m^2$ to 2.1 N/$m^2$. There are no areas within the immediate vicinity of the proposed causeway showing shear stress larger than the critical shear stress for average sediment gradings of 0.8 N/$m^2$. The only exception is at the very tip of the proposed causeway which is likely to be due to small scale bathymetric inaccuracies under the existing bridge.

In the vicinity of the Upper Harbour Bridge and proposed causeway, the spring flood tide shear stress ranges between:

- 0.4 N/$m^2$ in the channel to 2.1 N/$m^2$ near the western bridge abutments and western coastal boundaries, for the existing scenario (shown in Figure 3-11)
- 0.4 N/$m^2$ in the channel to 2.1 N/$m^2$ near the bridge abutments and western coastal boundaries, for the proposed scenario (shown in Figure 3-12).

Although this is essentially the same for both scenarios, the difference in shear stress is observed at several locations. This is shown in Figure 3-13, where:

- An increase in shear stress suggests increased erosion potential, this is shown near the tip of the proposed causeway and is up to 0.2 N/$m^2$, covering an area approximately 4,100 $m^2$
- A decrease in shear stress suggests increased deposition potential, this is shown immediately north and south of the proposed causeway tip and is up to -0.3 N/$m^2$, covering an area approximately 21,000 $m^2$.

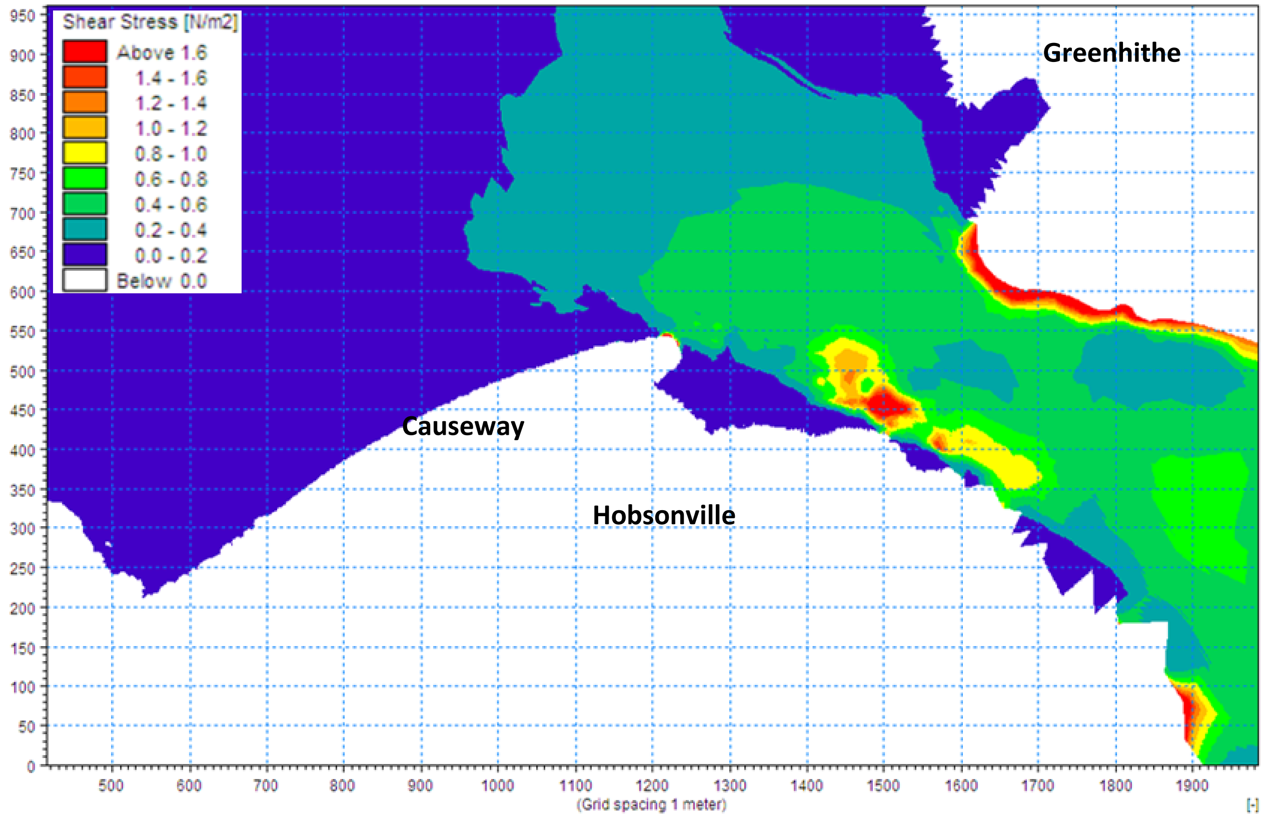


*Figure 3-11: Existing scenario – spring flood tide maximum shear stress*

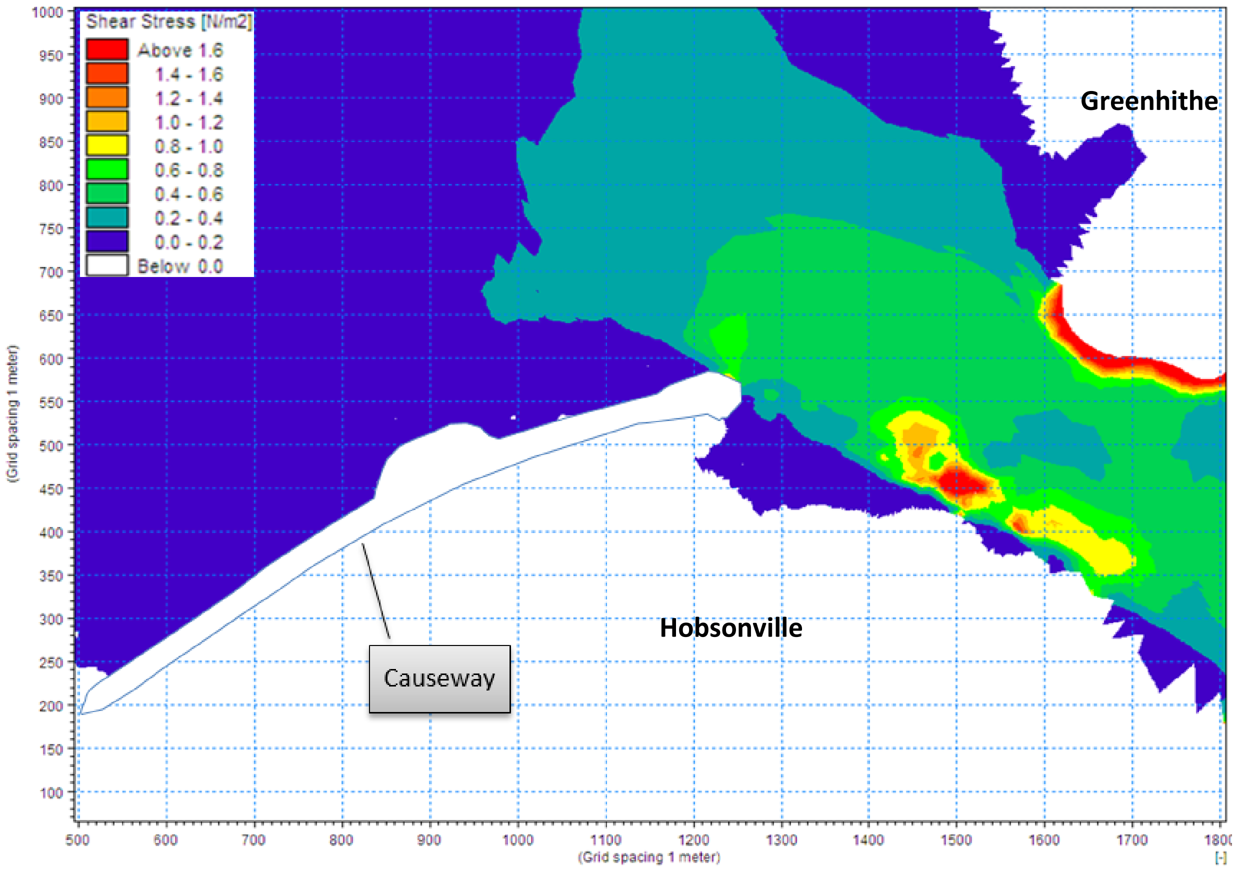


*Figure 3-12: Proposed scenario spring flood tide maximum shear stress*

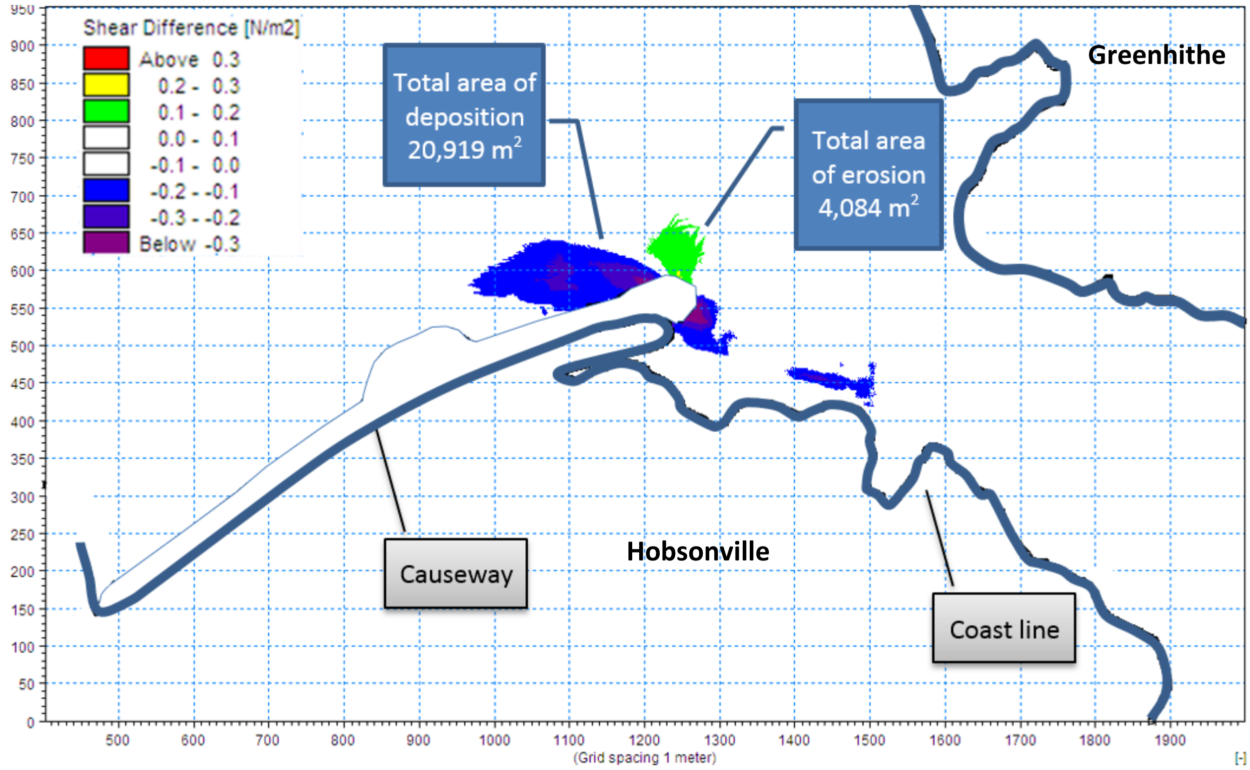


*Figure 3-13: Difference in shear stress for spring flood tide showing areas of potential erosion and deposition*

### 3.4.2 Shear stress – ebb tide

The shear stress for the existing situation is shown in Figure 3-14. Figure 3-15 shows the shear stress for the proposed development and Figure 3-16 shows the difference in shear stress. Shear stress was calculated using the formula in Section 3.4 for a typical spring ebb tide. There are no areas within the immediate vicinity of the proposed causeway showing shear stress larger than the critical shear stress of 0.8 N/m$^2$.

In the vicinity of the Upper Harbour Bridge and proposed causeway, the spring ebb tide shear stress ranges between:

- 0.4 N/m$^2$ in the channel to 1.8 N/m$^2$ near the western bridge abutments and western coastal boundaries, for the existing scenario (shown in Figure 3-14)
- 0.4 N/m$^2$ in the channel to 1.8 N/m$^2$ near the bridge abutments and western coastal boundaries, for the proposed scenario (shown in Figure 3-15).

Although this is essentially the same for both scenarios, the difference in shear stress is observed at several locations. This is shown in Figure 3-16, where:

- An increase in shear stress suggests increased erosion potential, this is shown near the tip of the proposed causeway and is up to 0.2 N/m$^2$, covering an area approximately 2,200 m$^2$
- A decrease in shear stress suggests increased deposition potential, this is shown immediately north and south of the proposed causeway tip and is up to -0.3 N/m$^2$, covering an area approximately 14,400 m$^2$.

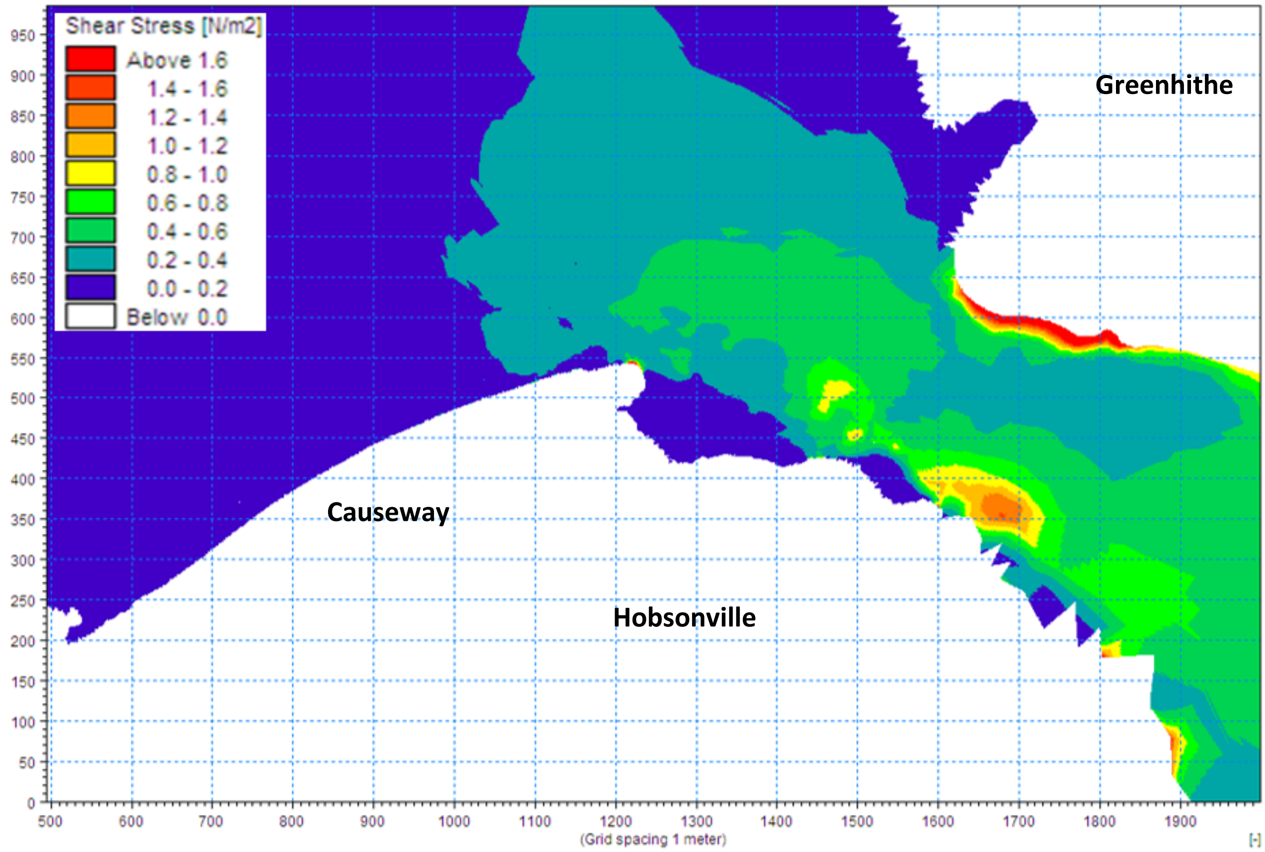


*Figure 3-14: Existing scenario - spring ebb tide maximum shear stress*

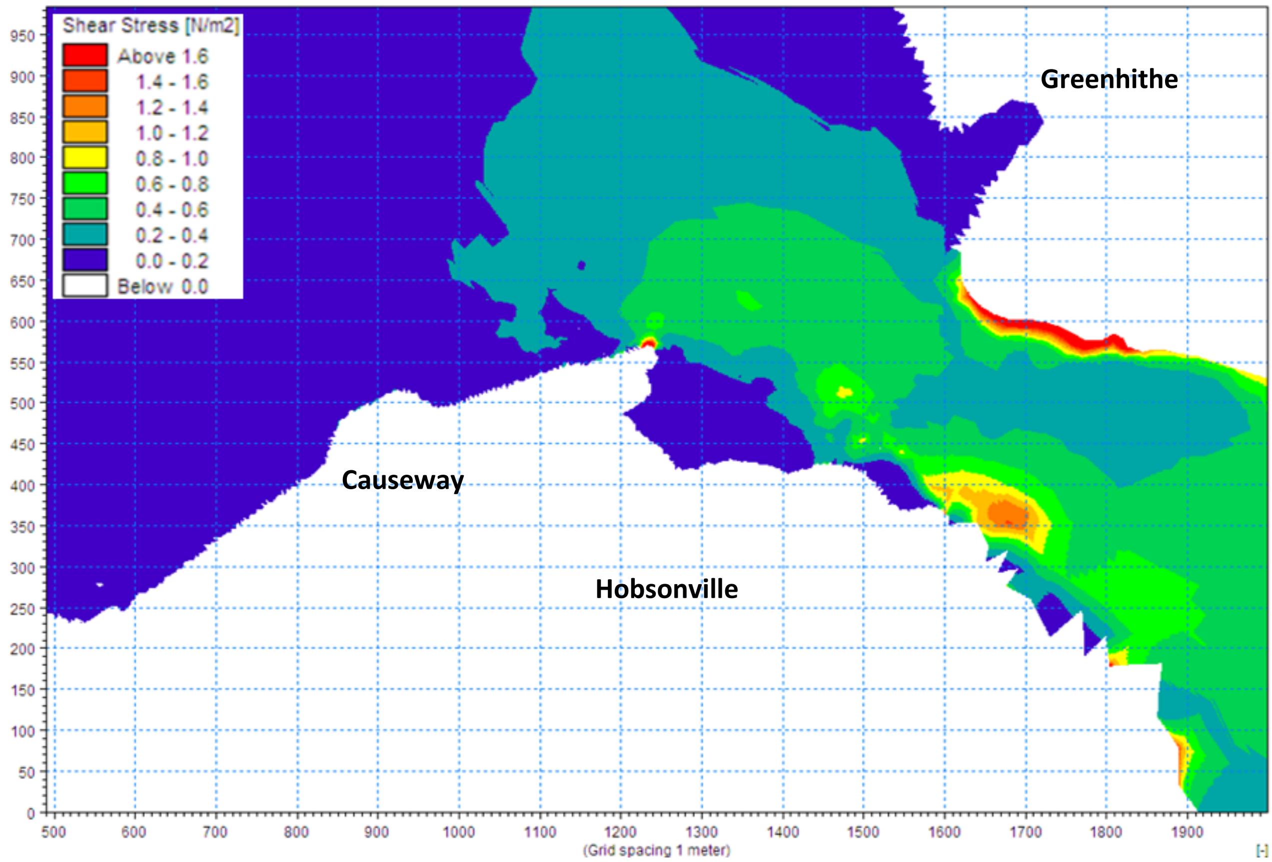


*Figure 3-15: Proposed scenario - spring ebb tide maximum shear stress*

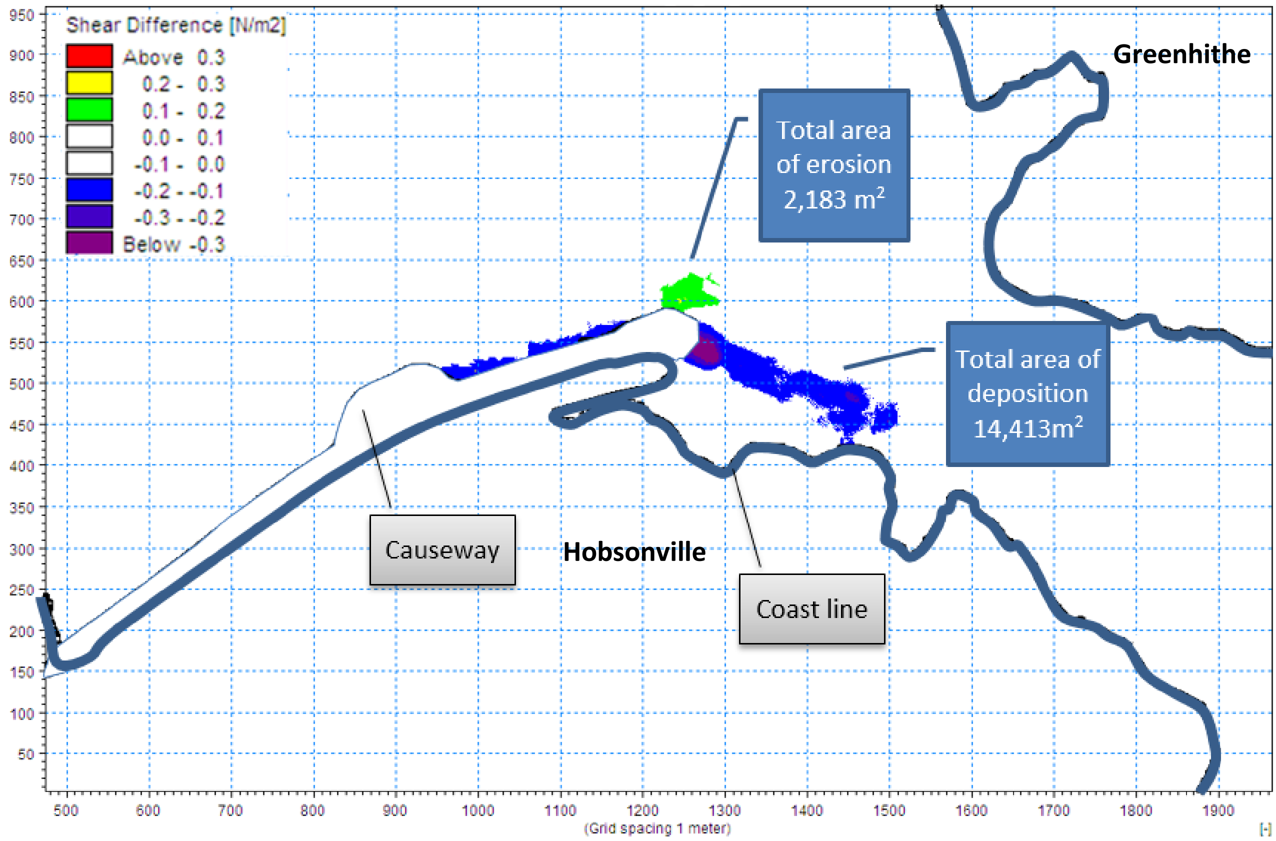


*Figure 3-16: Difference in shear stress for spring ebb tide showing areas of potential erosion and deposition*

# 4 Summary

The model built is based on data from the 2010 Waterview model (extended north using various data sets to include the Upper Harbour Bridge area and northern parts of the Waitemata Harbour).

Model validation shows a reasonable match between the Upper Harbour model and the Waterview model.

Model scenarios were:

1. Existing bathymetry with spring tide
2. Proposed bathymetry (causeway incorporated) with spring tide
3. Existing bathymetry with neap tide
4. Proposed bathymetry (causeway incorporated) with neap tide.

Results show that:

- the difference between the existing and proposed scenarios for a spring tide are:
  - surface elevation: between -0.0015 m and 0.001 m. These are very small changes and unlikely to be detectable
  - flood current speed: between -0.25 m/s (reduction in velocity) and 0.1 m/s (increase in velocity)
  - ebb current speed: between -0.25 m/s (reduction in velocity) and 0.1 m/s (increase in velocity)
- the differences in current speed between the proposed and existing scenarios for flood tide at representative locations (under the bridge and in the vicinity of the causeway) are:
  - 0.087 m/s near the proposed causeway and 0.007 m/s under the Upper Harbour Bridge for the spring tide. These results show away from the immediate vicinity of the causeway extension there is only minor changes in tidal velocity during spring tide.
  - 0.038 m/s near the proposed causeway and 0.003 m/s under the Upper Harbour Bridge for the neap tide. Due to the lower currents during neap tide changes are less than for spring tide.
- the differences in current speed between the proposed and existing scenarios for ebb tide at representative locations under the bridge and in the vicinity of the causeway are:
  - 0.086 m/s near the proposed causeway and 0.016 m/s under the Upper Harbour Bridge for the spring tide. These changes are in a similar order of magnitude near the causeway, but identify slightly greater effects during ebb tide than observed for flood tide under the bridge.
  - 0.041 m/s near the proposed causeway and 0.01 m/s under the Upper Harbour Bridge for the neap tide. These changes are in a similar order of magnitude near the causeway, but identify slightly greater effects during ebb tide than observed for flood tide under the bridge.
- The shear stress analysis shows that the effect of the velocity changes are only detectable in changing the existing seabed erosion potential in the immediate vicinity of causeway, but even at these locations the change in shear stress is small and that due to reductions in velocity any sediment eroded is likely to be deposited along the channel or intertidal areas immediately adjacent to the causeway.
- For a typical spring flood tide, shear stress analysis shows that :
  - the area of deposition potential is approximately 21,000 $m^2$
  - the area of erosion potential is approximately 4,100 $m^2$.

- for a typical spring ebb tide, shear stress analysis shows that :
  - the area of deposition potential is approximately 14,400 $m^2$
  - the area of erosion potential is approximately 2,200 $m^2$.

# 5 Conclusion

The hydrodynamic model study examining changes to tidal currents and erosion potential has shown that the proposed causeway widening and extension has minor and localised effects on tidal currents and shear stress. The causeway widening has no discernible effect on coastal processes while the extension will result in localised small scale erosion in the vicinity of the causeway extension and deposition in the areas of reduced tidal currents within the intertidal area adjacent to the causeway and along the south-western bank of the main channel.

# 6 Applicability

This report has been prepared for the benefit of Watercare Services Ltd with respect to the particular brief given to us to support an Assessment of Environmental Effects to assist in characterising the existing coastal environment and understand the potential effects of widening and extending the existing causeway. It may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

| Report prepared by: | Authorised for Tonkin & Taylor by: |
|---|---|
| ........................................................ | ........................................................ |
| Sarah Basheer | Richard Reinen-Hamill |
| Water Resources Engineer | Project Director |

SGB; RRH
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# 7 References

Guo, J. 2002. Hunter Rouse and Shields diagram. Proceedings of the 13th International Association of Hydraulic Research. Asian and Pacific Division Congress, Singapore, Malaysia, August 6-8.

Van Rijn, L.C. (1993). Principles of sediment transport in rivers, estuaries and coastal seas. Aqua publications, the Netherlands.

Tonkin & Taylor, NIWA (2010) Western Ring Route – Waterview Connection, G.14 Assessment of Coastal Processes.

## Appendix A: Data Point Tables for maximum current speed at flood and ebb tides with corresponding depth and surface elevation

**Table A1.1: Data point locations**

| | | |
|---|---|---|
| Data Point 1 | Easting | Northing |
| Data Point 2 | 1748582 | 5927679 |
| Data Point 3 | 1748643 | 5927705 |
| Data Point 4 | 1748688 | 5927664 |
| Data Point 5 | 1748714 | 5927721 |
| Data Point 6 | 1748831 | 5927766 |
| Data Point 7 | 1748908 | 5927805 |
| Data Point 8 | 1748986 | 5927856 |
| Data Point 9 | 1748830 | 5928087 |
| Data Point 10 | 1748739 | 5927959 |
| Data Point 11 | 1748671 | 5927868 |

**Table A1.2: Surface elevation at maximum current speed – spring tide (flood and ebb)**

| | **Spring tide surface elevation at maximum current speed for flood and ebb tides (mRL)** | | | | | |
|---|---|---|---|---|---|---|
| | Spring flood existing | Spring flood proposed | Difference in flood tide | Spring ebb existing | Spring ebb proposed | Difference in ebb tide |
| Data Point 1 | 0.893 | 0.891 | -0.002 | 0.991 | 0.993 | 0.002 |
| Data Point 2 | 0.891 | 0.887 | -0.004 | 0.988 | 0.984 | -0.004 |
| Data Point 3 | 0.892 | 0.895 | 0.003 | 0.987 | 0.984 | -0.002 |
| Data Point 4 | 0.891 | 0.891 | 0.000 | 0.987 | 0.985 | -0.002 |
| Data Point 5 | 0.891 | 0.891 | 0.000 | 0.985 | 0.984 | -0.001 |
| Data Point 6 | 0.891 | 0.891 | 0.000 | 0.985 | 0.984 | -0.001 |
| Data Point 7 | 0.887 | 0.887 | 0.000 | 0.992 | 0.991 | 0.000 |
| Data Point 8 | 0.894 | 0.894 | 0.000 | 0.991 | 0.990 | 0.000 |
| Data Point 9 | 0.891 | 0.891 | 0.000 | 0.991 | 0.990 | 0.000 |
| Data Point 10 | 0.891 | 0.891 | -0.001 | 0.991 | 0.990 | 0.000 |
| Data Point 11 | 0.894 | 0.893 | -0.001 | 0.993 | 0.993 | 0.000 |

**Table A1.3: Maximum current speed – spring tide (flood and ebb)**

| | Spring tide maximum current speed for flood and ebb tides (m/s) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Spring flood existing | Spring flood proposed | Difference in flood tide | | Spring ebb existing | Spring ebb proposed | Difference in ebb tide | |
| Data Point 1 | 0.368 | 0.1 | -0.268 | -72.8% | 0.351 | 0.292 | -0.059 | -16.8% |
| Data Point 2 | 0.478 | 0.565 | 0.087 | 18.2% | 0.462 | 0.548 | 0.086 | 18.6% |
| Data Point 3 | 0.49 | 0.442 | -0.049 | -10.0% | 0.43 | 0.361 | -0.069 | -16.0% |
| Data Point 4 | 0.482 | 0.501 | 0.02 | 4.1% | 0.483 | 0.52 | 0.037 | 7.7% |
| Data Point 5 | 0.497 | 0.504 | 0.007 | 1.4% | 0.464 | 0.48 | 0.016 | 3.4% |
| Data Point 6 | 0.44 | 0.447 | 0.007 | 1.6% | 0.405 | 0.417 | 0.012 | 3.0% |
| Data Point 7 | 0.166 | 0.171 | 0.005 | 3.0% | 0.198 | 0.207 | 0.008 | 4.0% |
| Data Point 8 | 0.126 | 0.134 | 0.008 | 6.3% | 0.05 | 0.055 | 0.005 | 10.0% |
| Data Point 9 | 0.325 | 0.346 | 0.021 | 6.5% | 0.344 | 0.358 | 0.014 | 4.1% |
| Data Point 10 | 0.406 | 0.43 | 0.024 | 5.9% | 0.395 | 0.41 | 0.015 | 3.8% |
| Data Point 11 | 0.382 | 0.424 | 0.043 | 11.3% | 0.344 | 0.343 | 0 | 0.0% |

**Table A1.4: Water depth at maximum current speed - spring tide (flood and ebb)**

| | **Spring Tide Total water depth at maximum speed for flood and ebb tides (m)** | | | | | |
|---|---|---|---|---|---|---|
| | Spring flood existing | Spring flood proposed | Difference in flood tide | Spring ebb existing | Spring ebb proposed | Difference in ebb tide |
| Data Point 1 | 5.608 | 5.583 | -0.024 | 5.707 | 5.686 | -0.020 |
| Data Point 2 | 6.755 | 6.759 | 0.004 | 6.852 | 6.856 | 0.004 |
| Data Point 3 | 7.255 | 7.253 | -0.002 | 7.350 | 7.342 | -0.008 |
| Data Point 4 | 6.958 | 6.957 | -0.001 | 7.054 | 7.051 | -0.003 |
| Data Point 5 | 7.379 | 7.387 | 0.008 | 7.473 | 7.480 | 0.007 |
| Data Point 6 | 6.388 | 6.396 | 0.008 | 6.482 | 6.490 | 0.008 |
| Data Point 7 | 3.104 | 3.104 | 0.000 | 3.208 | 3.208 | 0.000 |
| Data Point 8 | 3.143 | 3.092 | -0.052 | 3.240 | 3.189 | -0.052 |
| Data Point 9 | 4.758 | 4.758 | 0.000 | 4.858 | 4.857 | 0.000 |
| Data Point 10 | 7.205 | 7.205 | 0.000 | 7.305 | 7.305 | 0.000 |
| Data Point 11 | 8.265 | 8.272 | 0.008 | 8.364 | 8.373 | 0.009 |

**Table A1.5: Surface elevation at maximum current speed – neap tide (flood and ebb)**

| | **Neap Tide Surface Elevation at maximum current speed for flood and ebb tides (mRL)** | | | | | |
|---|---|---|---|---|---|---|
| | Neap flood existing | Neap flood proposed | Difference in flood tide | Neap ebb existing | Neap ebb proposed | Difference in ebb tide |
| Data Point 1 | 0.667 | 0.666 | 0.000 | 0.347 | 0.348 | 0.001 |
| Data Point 2 | 0.666 | 0.665 | -0.001 | 0.346 | 0.345 | -0.001 |
| Data Point 3 | 0.667 | 0.667 | 0.001 | 0.346 | 0.345 | -0.001 |
| Data Point 4 | 0.666 | 0.666 | 0.000 | 0.346 | 0.345 | -0.001 |
| Data Point 5 | 0.666 | 0.666 | 0.000 | 0.346 | 0.345 | 0.000 |
| Data Point 6 | 0.666 | 0.666 | 0.000 | 0.346 | 0.345 | 0.000 |
| Data Point 7 | 0.665 | 0.665 | 0.000 | 0.348 | 0.348 | 0.000 |
| Data Point 8 | 0.667 | 0.667 | 0.000 | 0.348 | 0.348 | 0.000 |
| Data Point 9 | 0.666 | 0.666 | 0.000 | 0.348 | 0.348 | 0.000 |
| Data Point 10 | 0.666 | 0.666 | 0.000 | 0.347 | 0.347 | 0.000 |
| Data Point 11 | 0.667 | 0.667 | 0.000 | 0.348 | 0.348 | 0.000 |

**Table A1.6: Maximum current speed – neap tide (flood and ebb)**

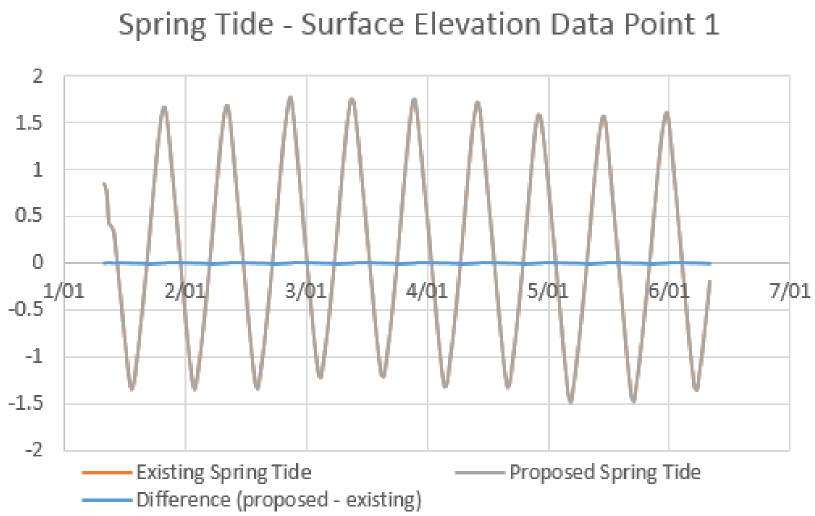
| | **Neap tide maximum current speed for flood and ebb tides (m/s)** |
|---|---|

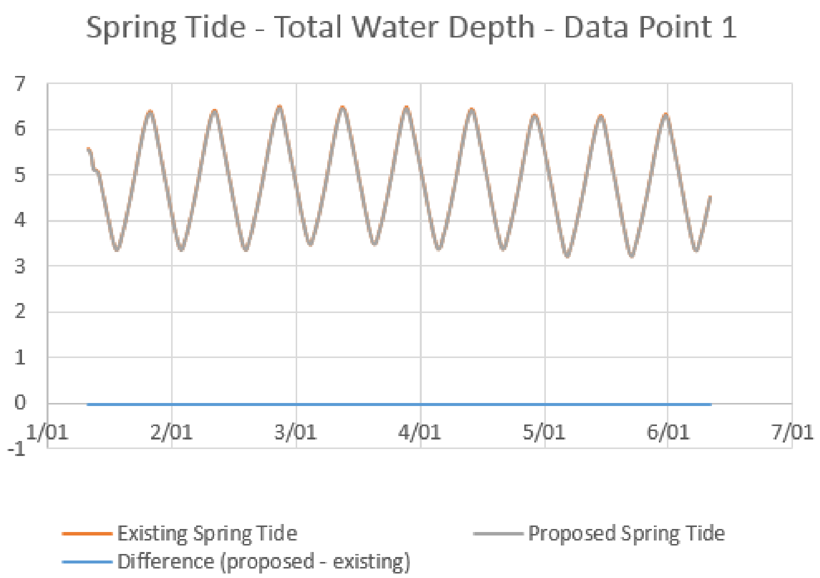
| | Neap flood existing | Neap flood proposed | Difference in flood tide | | Neap ebb existing | Neap ebb proposed | Difference in ebb tide | |
|---|---|---|---|---|---|---|---|---|
| Data Point 1 | 0.175 | 0.039 | -0.136 | -77.7% | 0.171 | 0.131 | -0.04 | -23.4% |
| Data Point 2 | 0.234 | 0.272 | 0.038 | 16.2% | 0.245 | 0.285 | 0.041 | 16.7% |
| Data Point 3 | 0.239 | 0.214 | -0.025 | -10.5% | 0.224 | 0.167 | -0.057 | -25.4% |
| Data Point 4 | 0.237 | 0.247 | 0.01 | 4.2% | 0.262 | 0.285 | 0.024 | 9.2% |
| Data Point 5 | 0.244 | 0.247 | 0.003 | 1.2% | 0.251 | 0.26 | 0.01 | 4.0% |
| Data Point 6 | 0.214 | 0.217 | 0.003 | 1.4% | 0.218 | 0.224 | 0.007 | 3.2% |
| Data Point 7 | 0.077 | 0.079 | 0.002 | 2.6% | 0.104 | 0.108 | 0.004 | 3.8% |
| Data Point 8 | 0.057 | 0.061 | 0.004 | 7.0% | 0.026 | 0.029 | 0.002 | 7.7% |
| Data Point 9 | 0.159 | 0.17 | 0.011 | 6.9% | 0.186 | 0.194 | 0.008 | 4.3% |
| Data Point 10 | 0.201 | 0.214 | 0.013 | 6.5% | 0.214 | 0.222 | 0.008 | 3.7% |
| Data Point 11 | 0.187 | 0.21 | 0.022 | 11.8% | 0.185 | 0.182 | -0.003 | -1.6% |

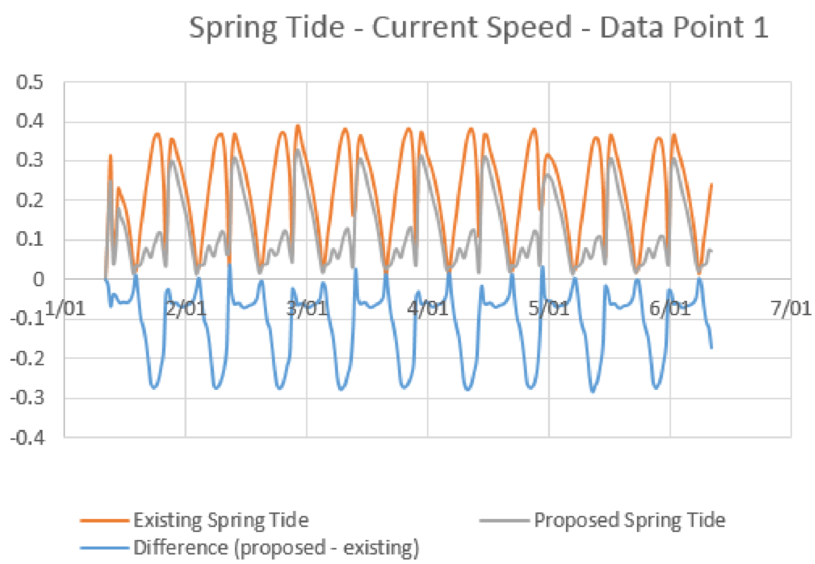
## Table A1.7: Water depth at maximum current speed - neap tide (flood and ebb)

| | **Neap tide total water depth at maximum speed for flood and ebb tides (m)** | | | | | |
|---|---|---|---|---|---|---|
| | Neap flood existing | Neap flood proposed | Difference in flood tide | Neap ebb existing | Neap ebb proposed | Difference in ebb tide |
| Data Point 1 | 5.382 | 5.359 | -0.023 | 5.063 | 5.041 | -0.022 |
| Data Point 2 | 6.530 | 6.537 | 0.007 | 6.210 | 6.218 | 0.007 |
| Data Point 3 | 7.030 | 7.025 | -0.005 | 6.709 | 6.703 | -0.007 |
| Data Point 4 | 6.733 | 6.732 | -0.001 | 6.413 | 6.411 | -0.002 |
| Data Point 5 | 7.154 | 7.162 | 0.008 | 6.834 | 6.841 | 0.008 |
| Data Point 6 | 6.163 | 6.171 | 0.008 | 5.843 | 5.851 | 0.008 |
| Data Point 7 | 2.881 | 2.881 | 0.000 | 2.564 | 2.564 | 0.000 |
| Data Point 8 | 2.916 | 2.865 | -0.052 | 2.597 | 2.546 | -0.052 |
| Data Point 9 | 4.533 | 4.533 | 0.000 | 4.214 | 4.214 | 0.000 |
| Data Point 10 | 6.980 | 6.981 | 0.000 | 6.661 | 6.662 | 0.001 |
| Data Point 11 | 8.038 | 8.046 | 0.009 | 7.719 | 7.728 | 0.009 |

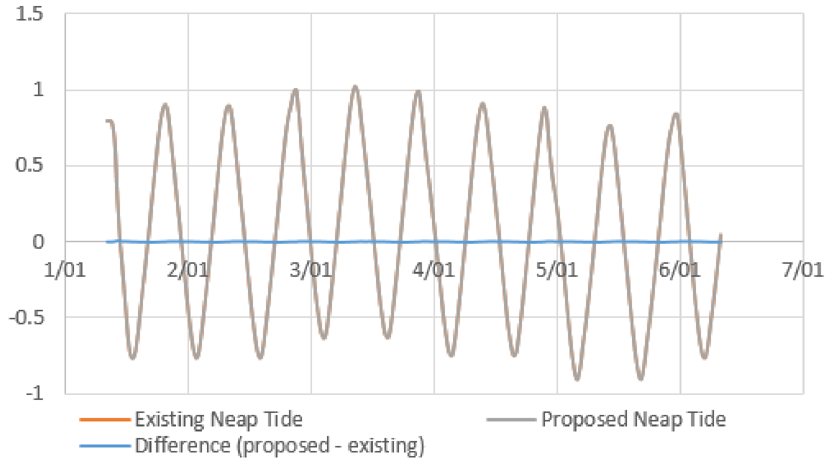
## Appendix B: Timeseries for surface elevation, current speed and total water depth at data point locations


Spring Tide - Surface Elevation Data Point 1
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

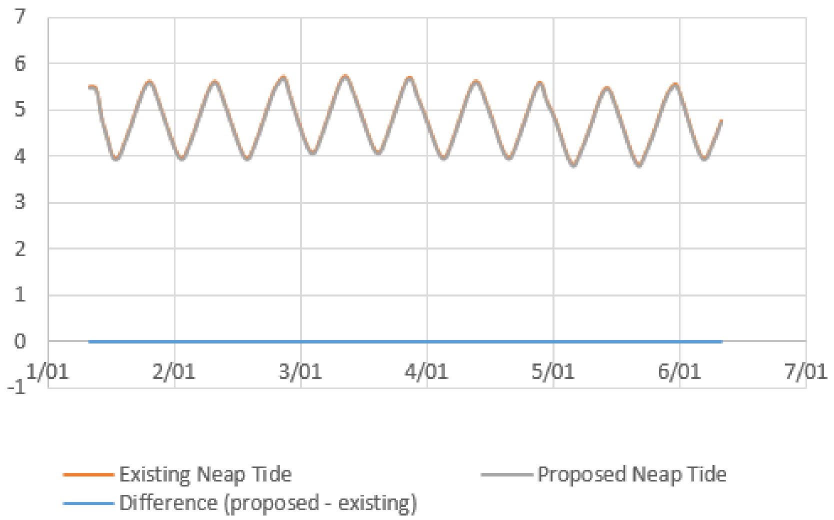

Spring Tide - Total Water Depth - Data Point 1
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Current Speed - Data Point 1
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

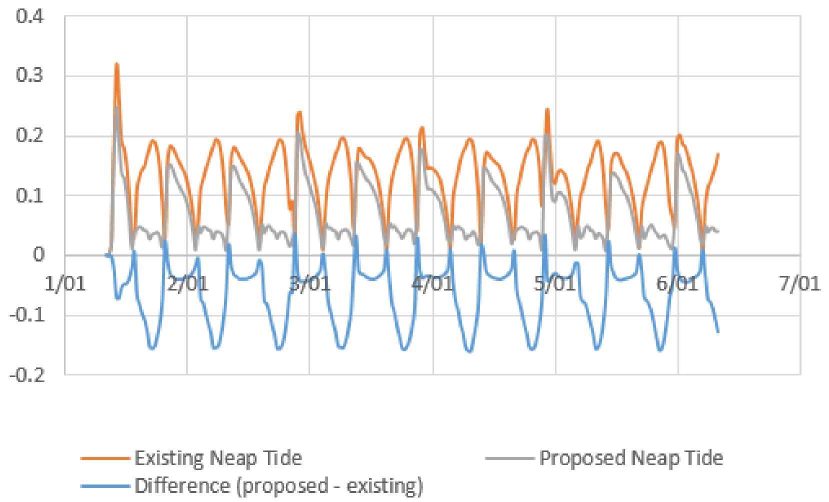
Neap Tide - Surface Elevation - Data Point 1



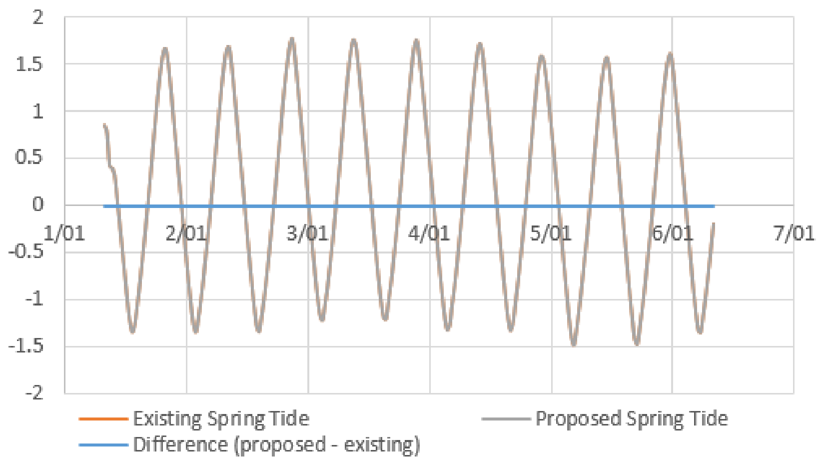

Neap Tide - Total Water Depth - Data Point 1



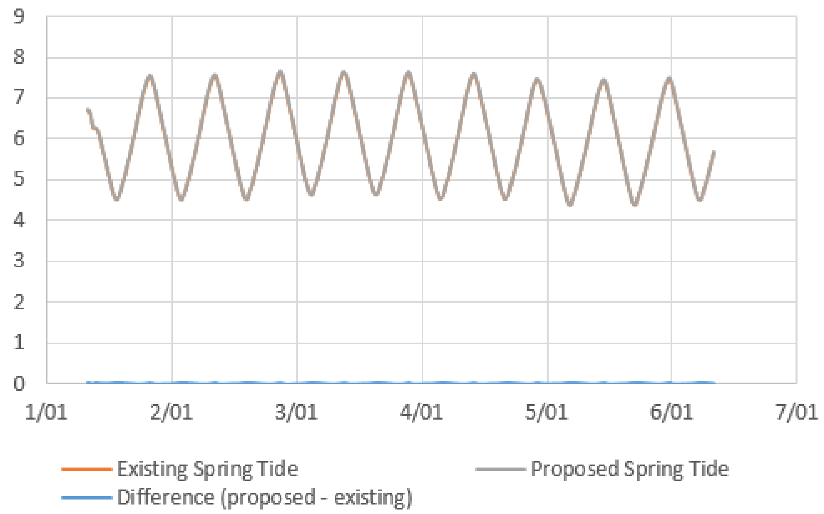

Neap Tide - Current Speed -Data Point 1



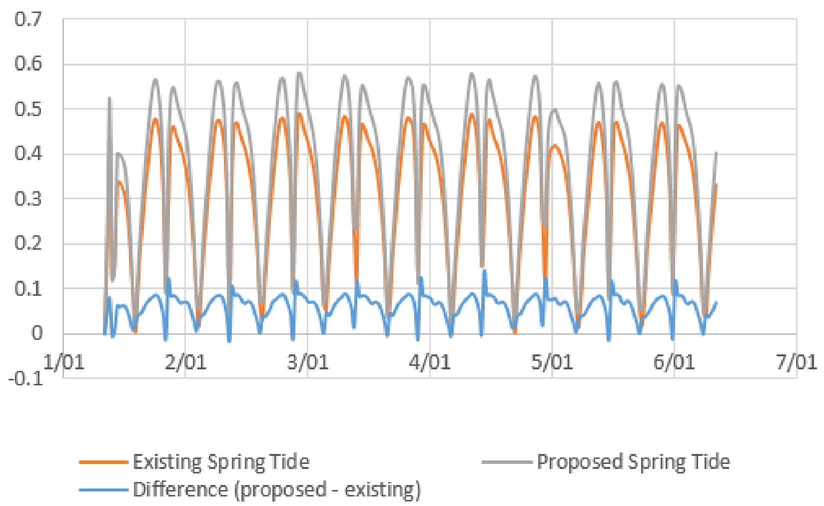
Spring Tide - Surface Elevation Data Point 2



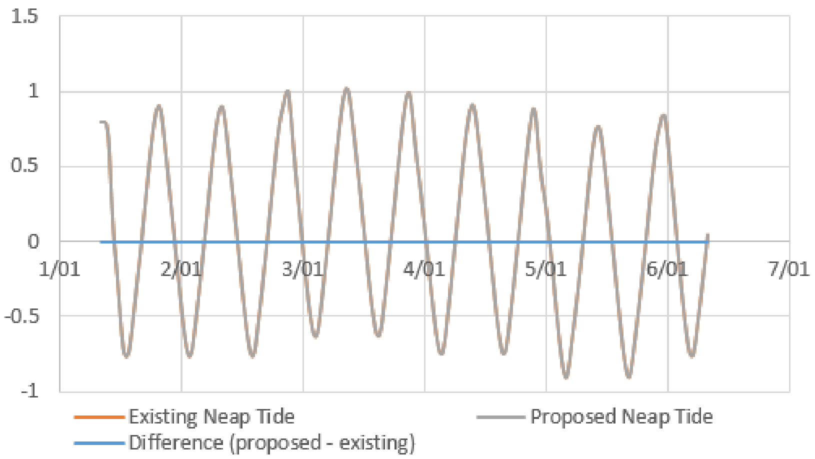

Spring Tide - Total Water Depth - Data Point 2



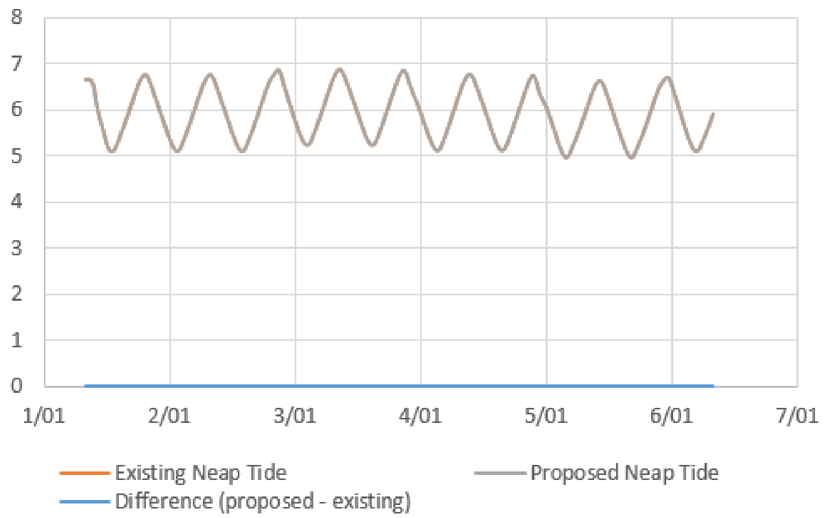

Spring Tide - Current Speed - Data Point 2



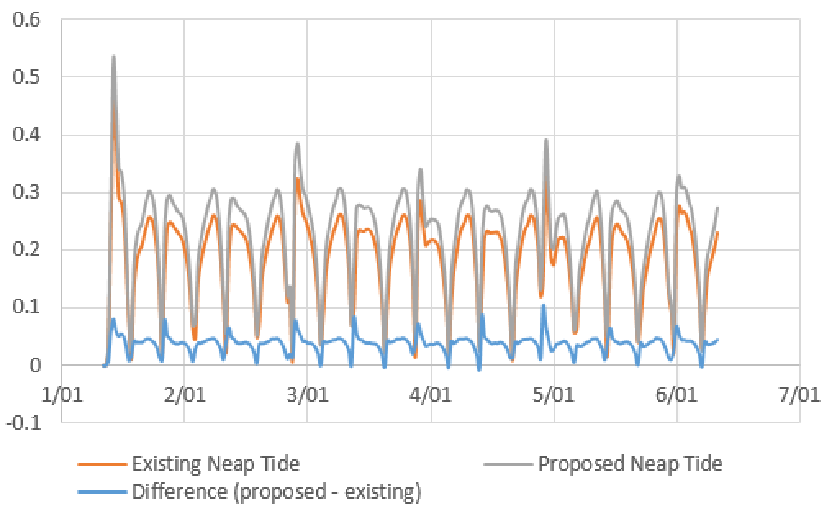
Neap Tide - Surface Elevation - Data Point 2

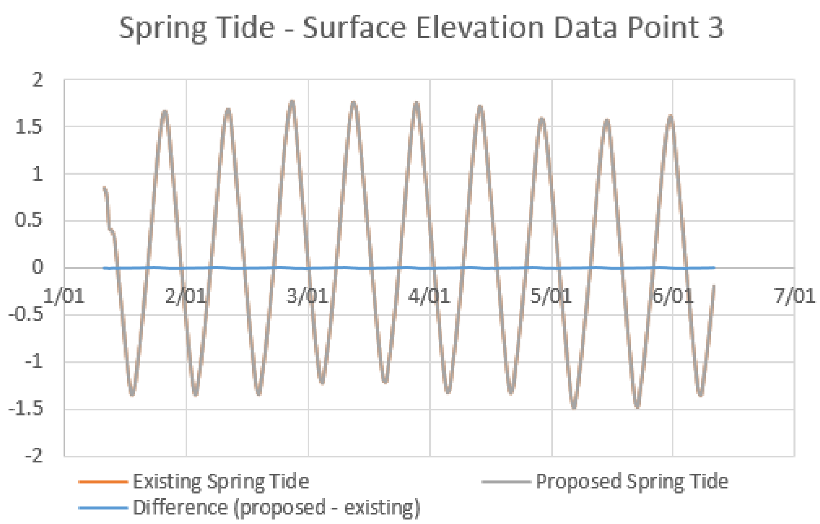


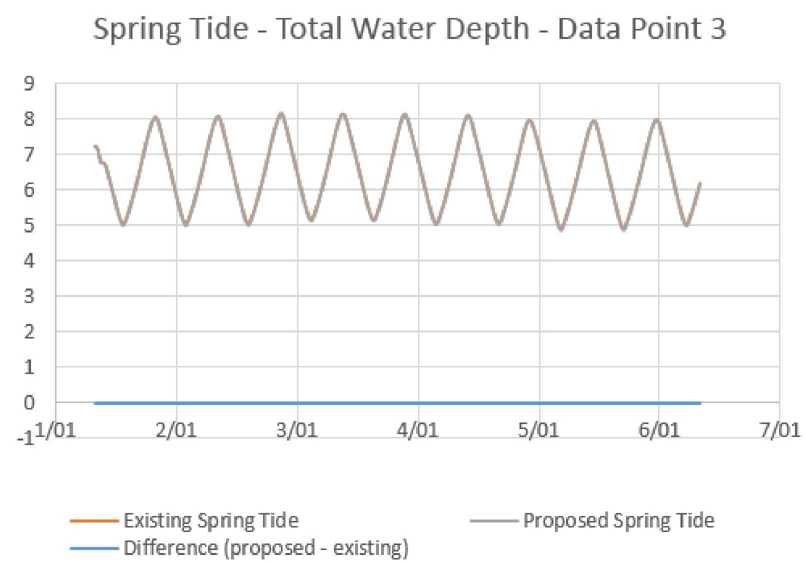

Neap Tide - Total Water Depth - Data Point 2

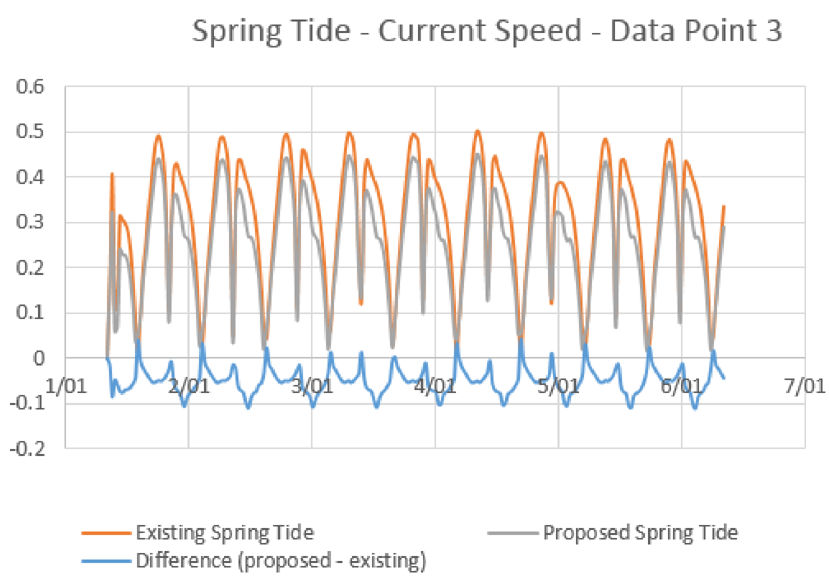


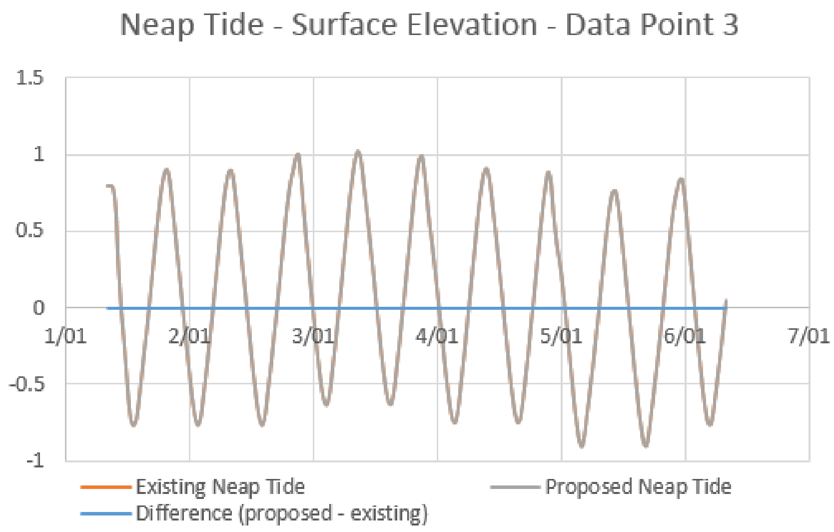

Neap Tide - Current Speed -Data Point 2

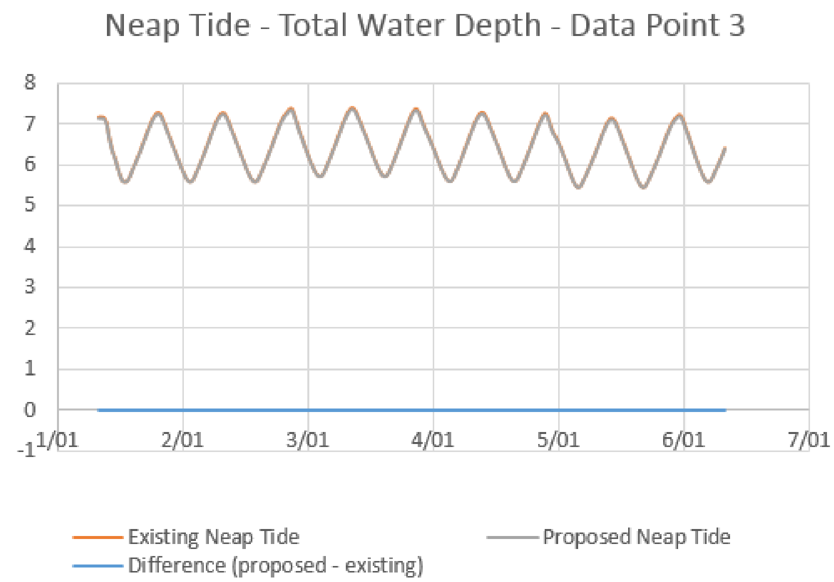


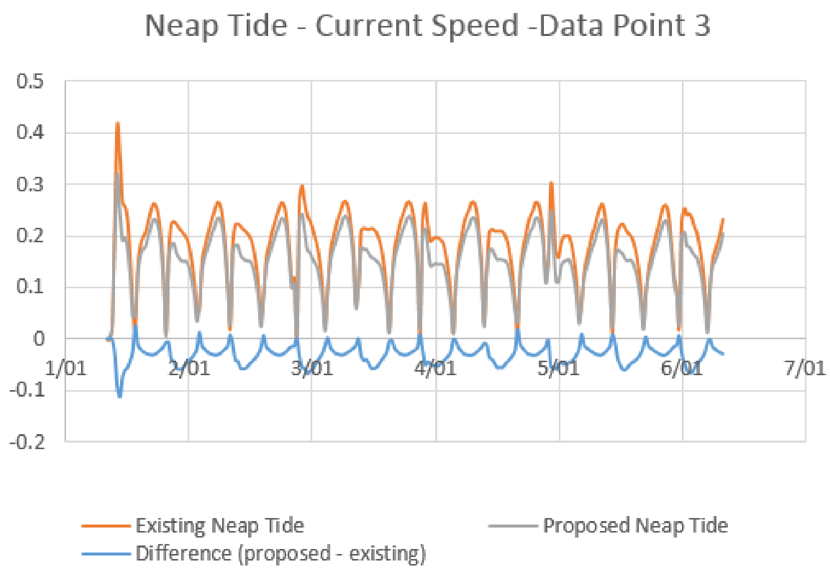

Spring Tide - Surface Elevation Data Point 3
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Total Water Depth - Data Point 3
9
8
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

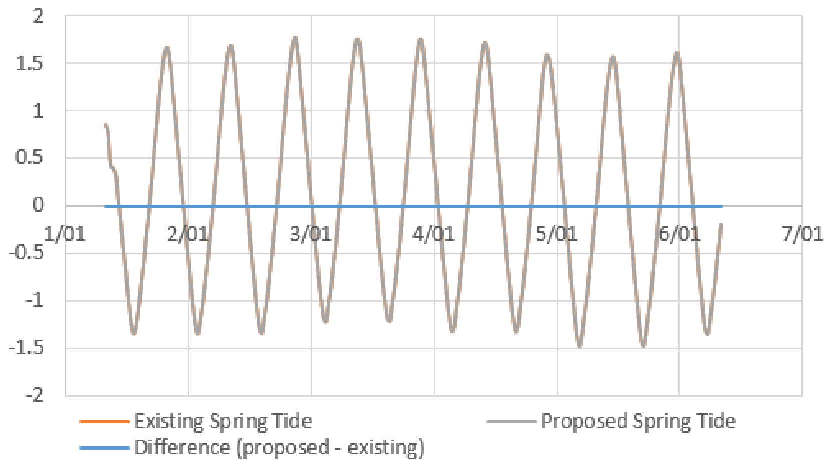

Spring Tide - Current Speed - Data Point 3
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Neap Tide - Surface Elevation - Data Point 3
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

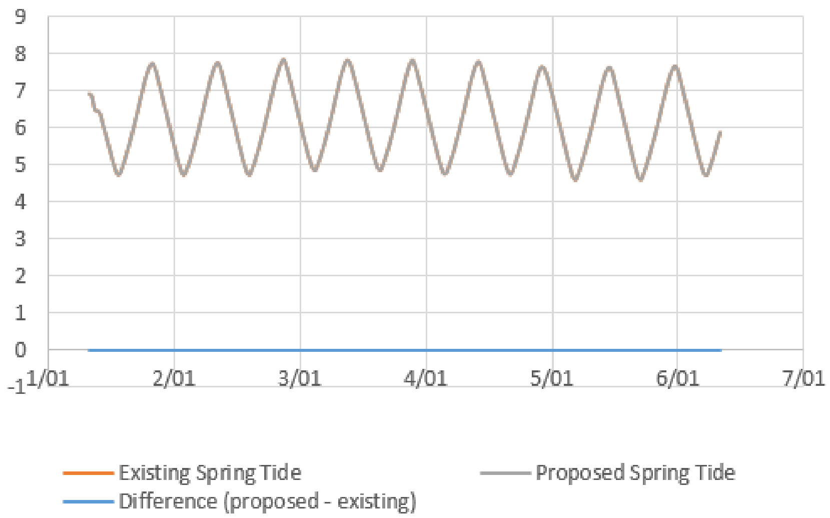

Neap Tide - Total Water Depth - Data Point 3
8
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Current Speed -Data Point 3
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

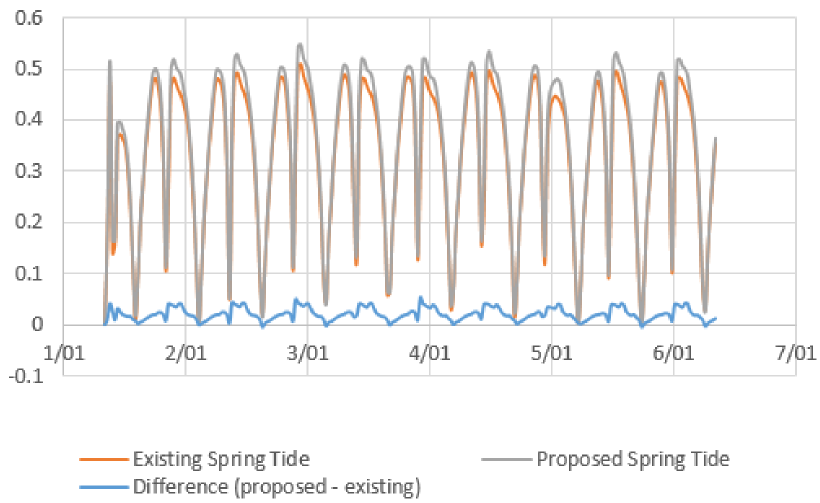
Spring Tide - Surface Elevation - Data Point 4

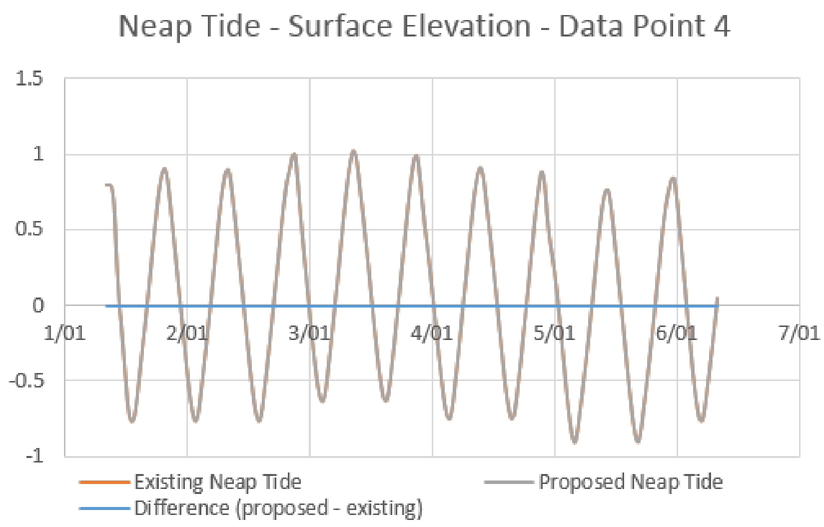


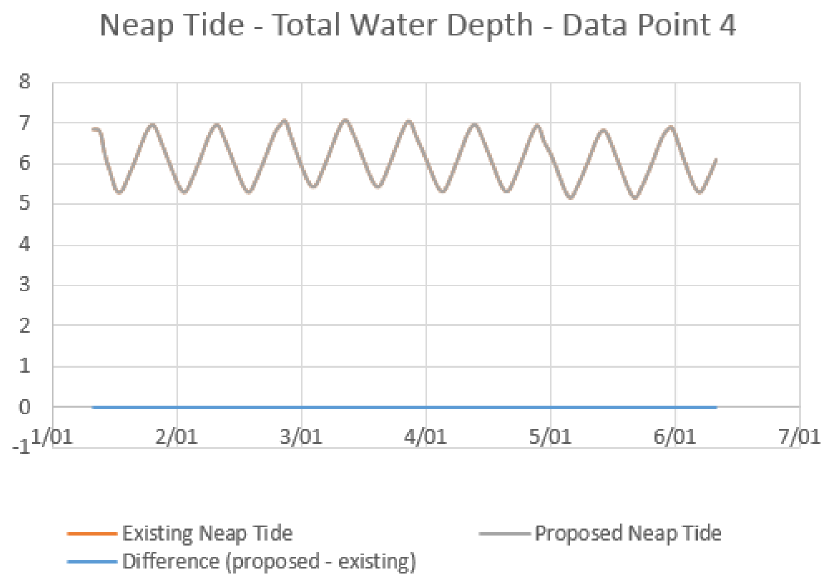

Spring Tide - Total Water Depth - Data Point 4

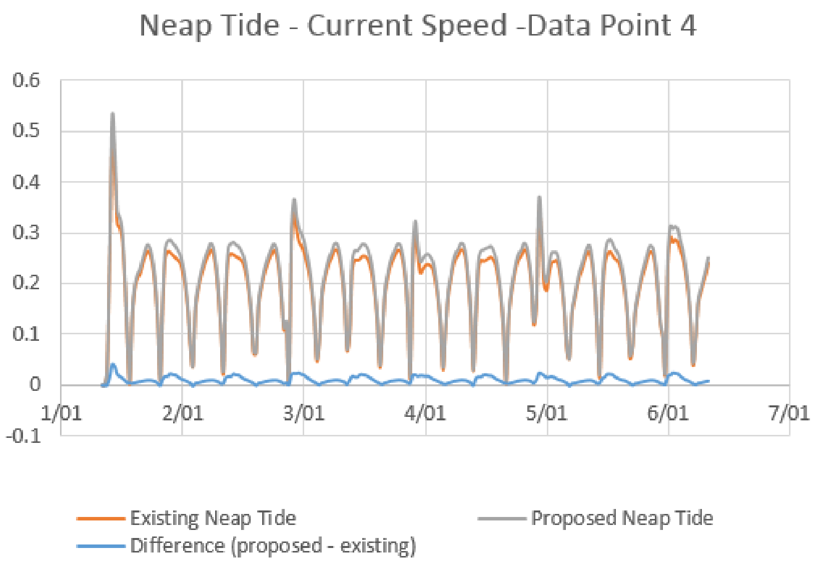


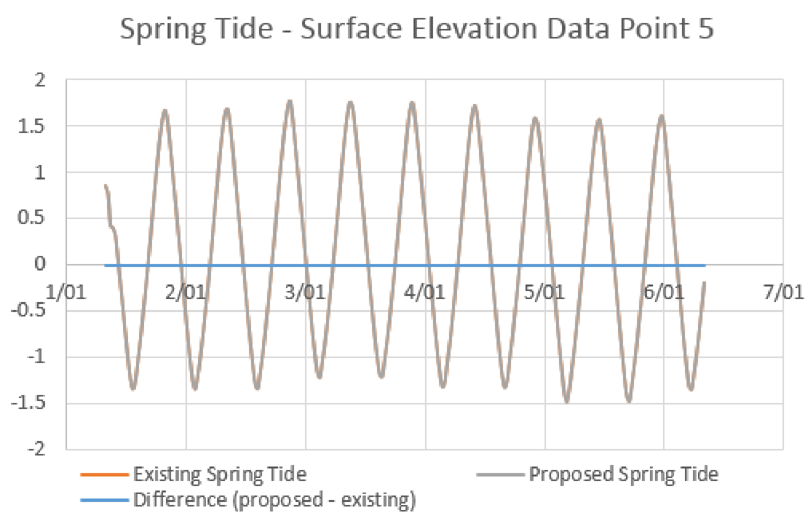

Spring Tide - Current Speed - Data Point 4

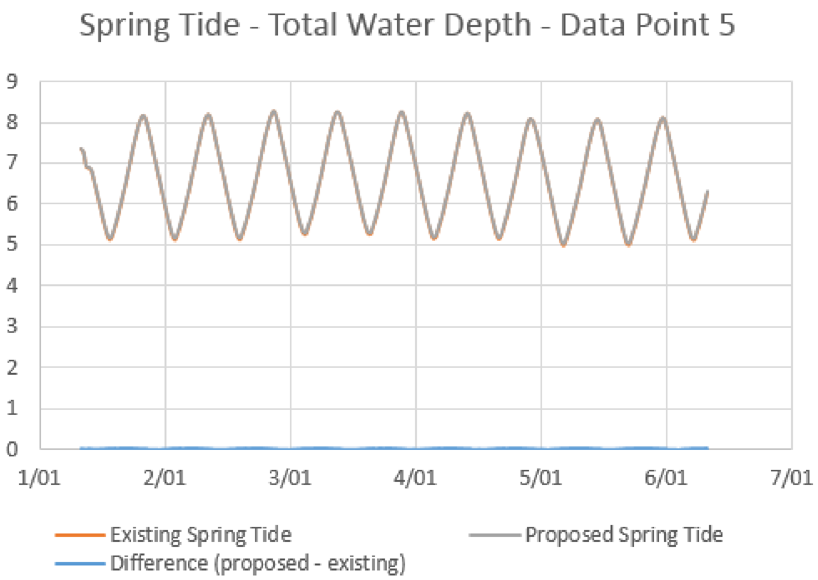


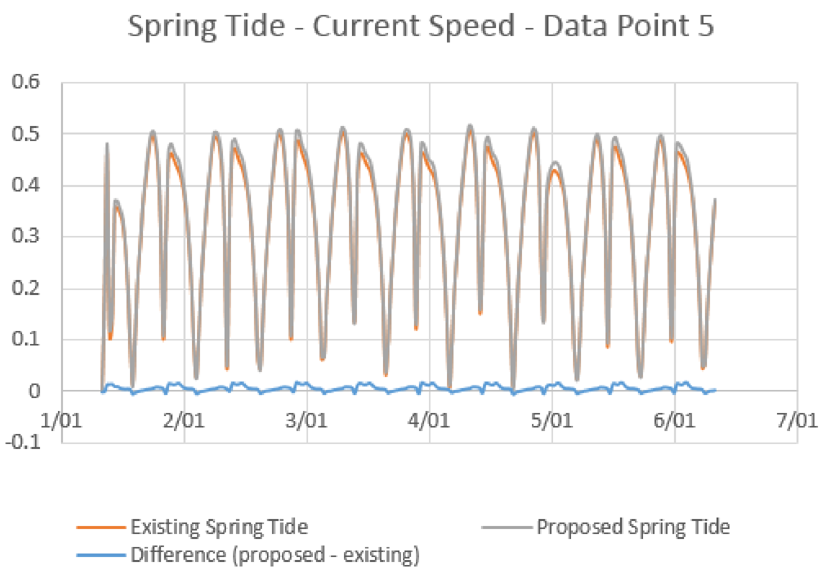

Neap Tide - Surface Elevation - Data Point 4
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Total Water Depth - Data Point 4
8
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

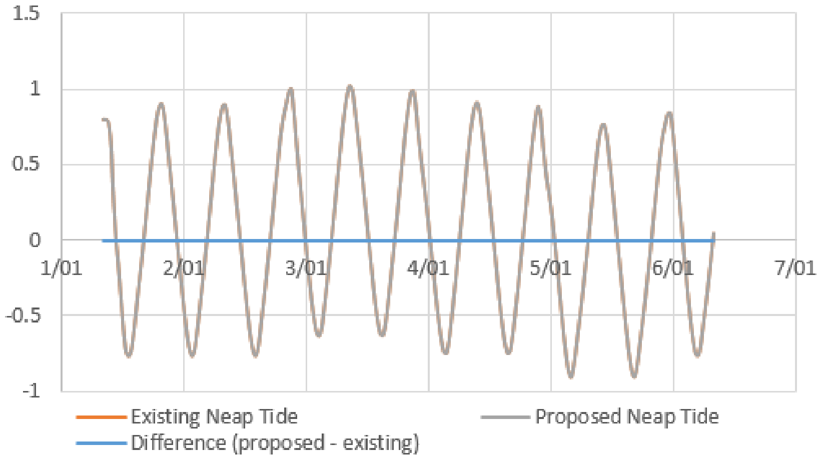

Neap Tide - Current Speed -Data Point 4
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Spring Tide - Surface Elevation Data Point 5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

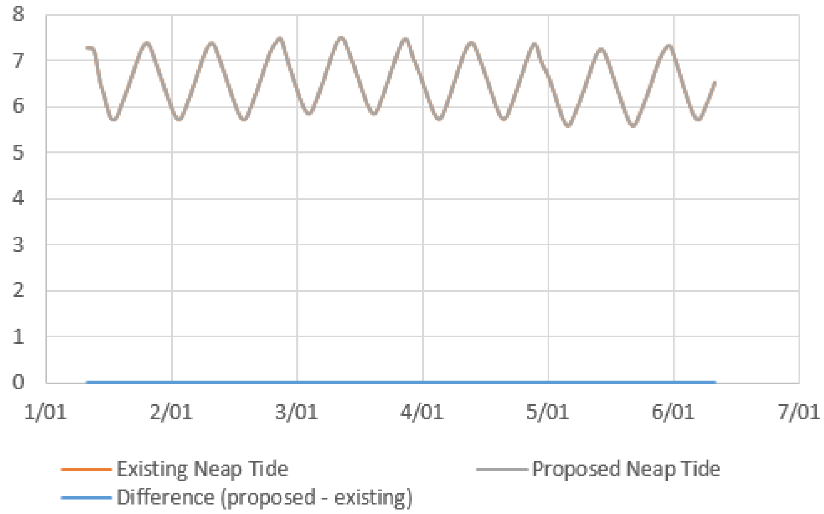

Spring Tide - Total Water Depth - Data Point 5
9
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Spring Tide - Current Speed - Data Point 5
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

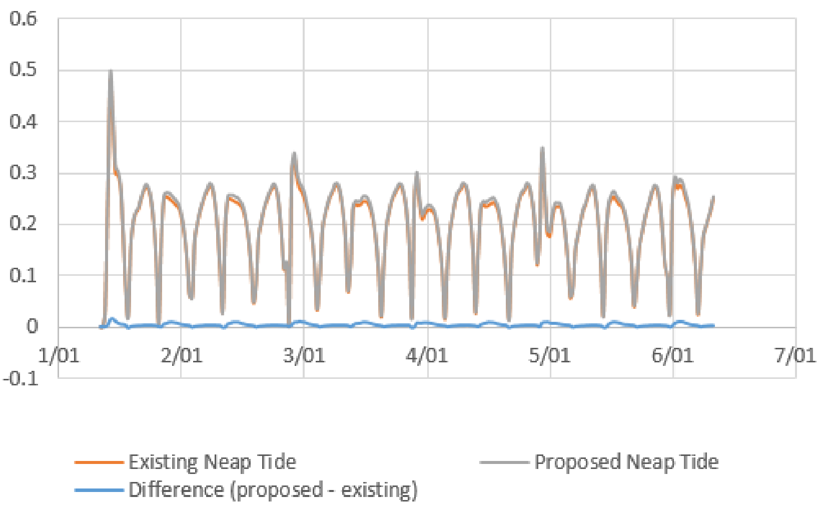
Neap Tide - Surface Elevation - Data Point 5



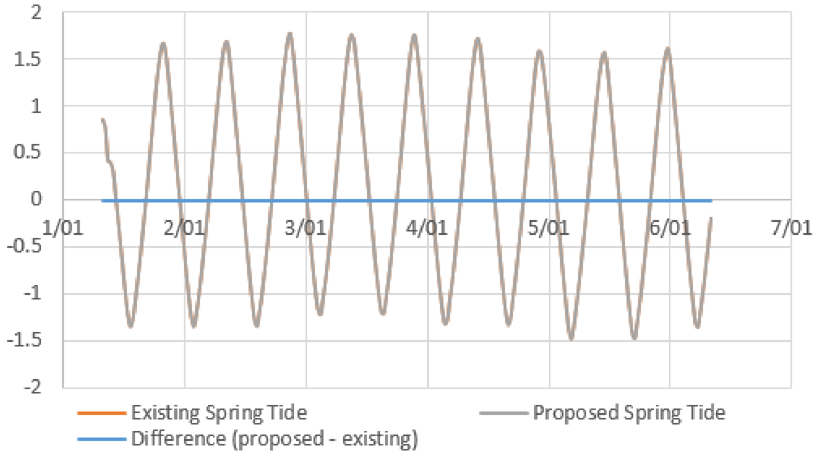

Neap Tide - Total Water Depth - Data Point 5



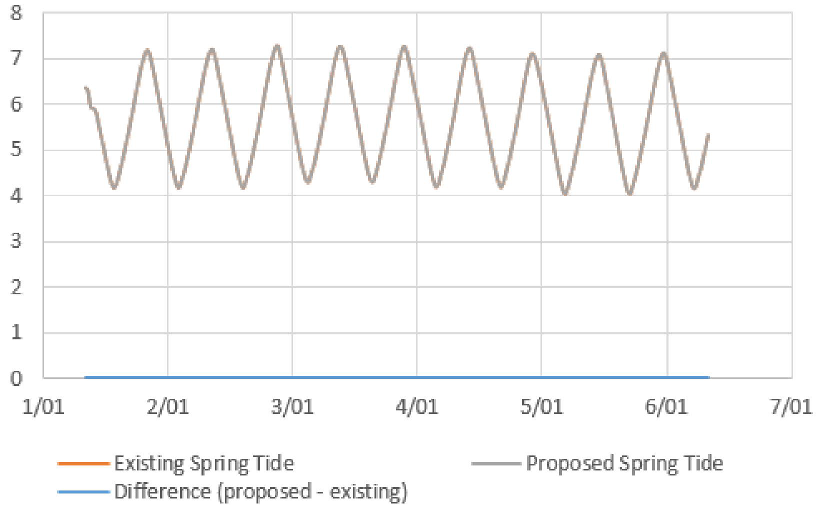

Neap Tide - Current Speed -Data Point 5



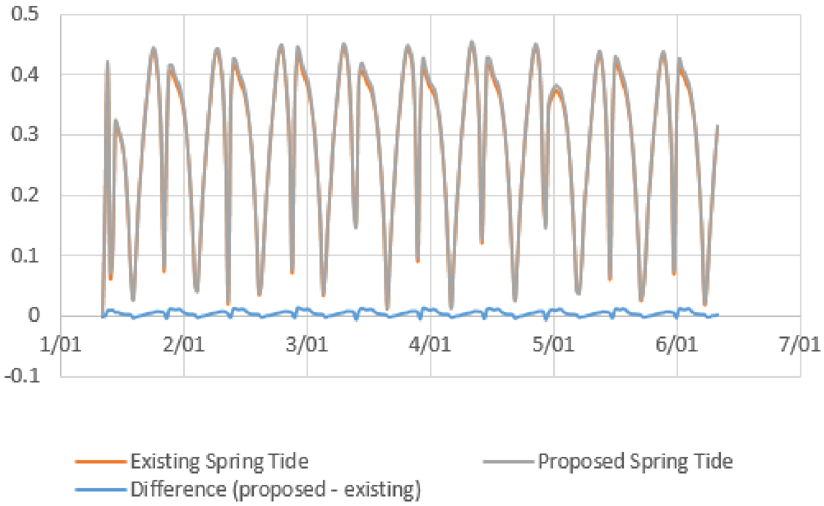
Spring Tide - Surface Elevation -Data Point 6



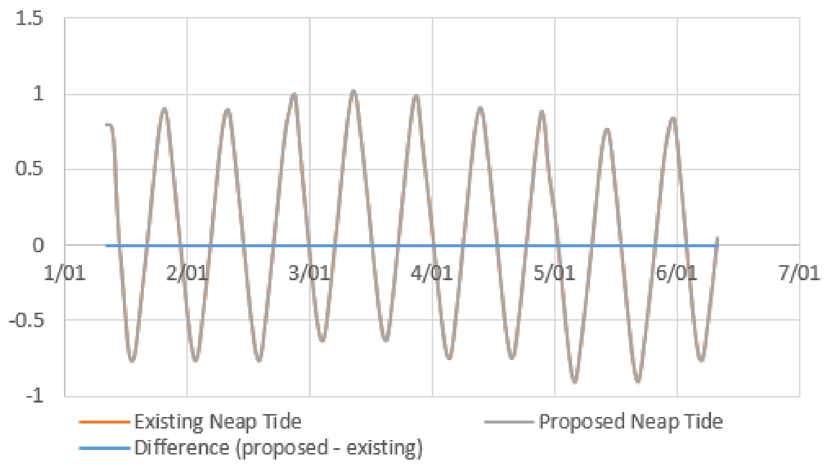

Spring Tide - Total Water Depth - Data Point 6



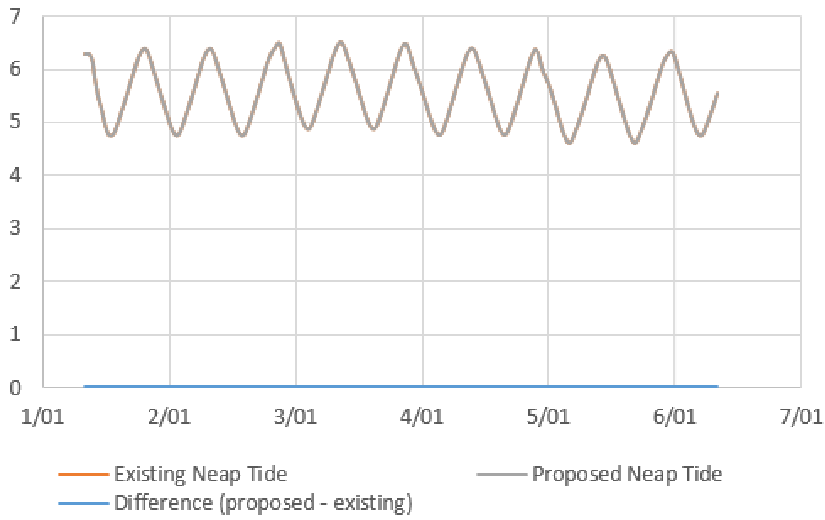

Spring Tide - Current Speed - Data Point 6



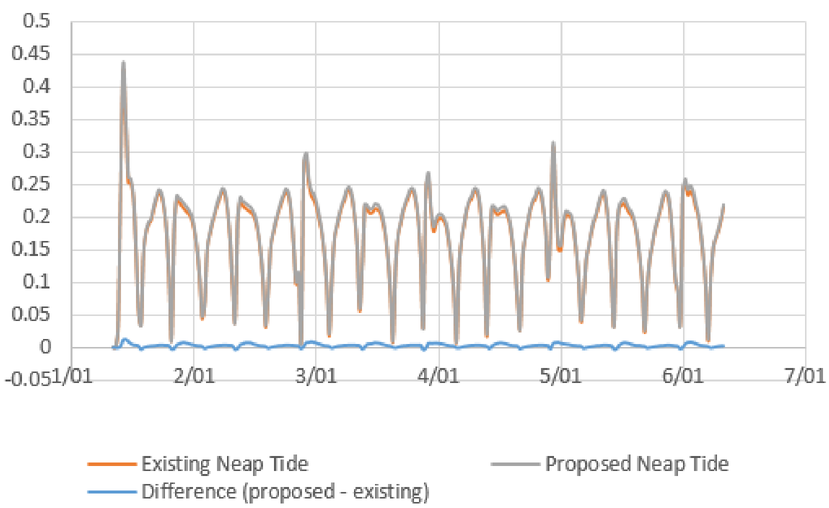
Neap Tide - Surface Elevation - Data Point 6



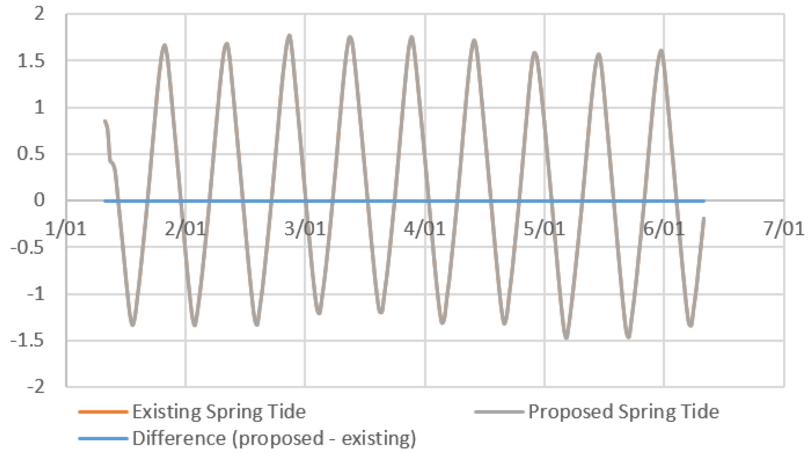

Neap Tide - Total Water Depth - Data Point 6



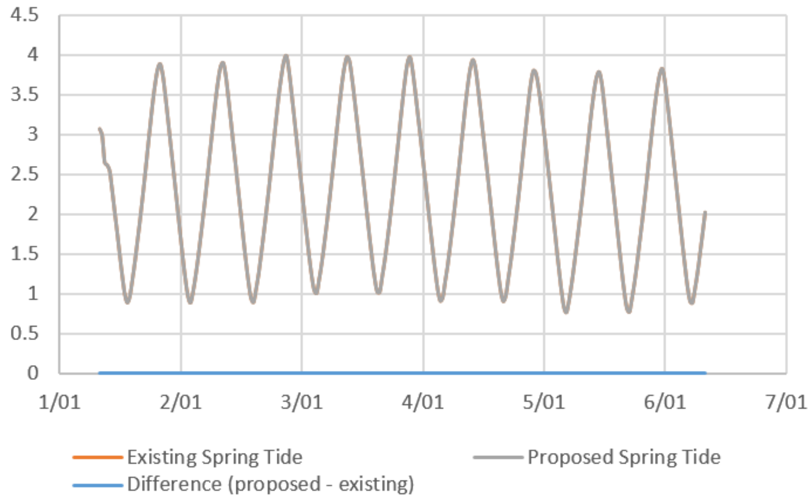

Neap Tide - Current Speed -Data Point 6



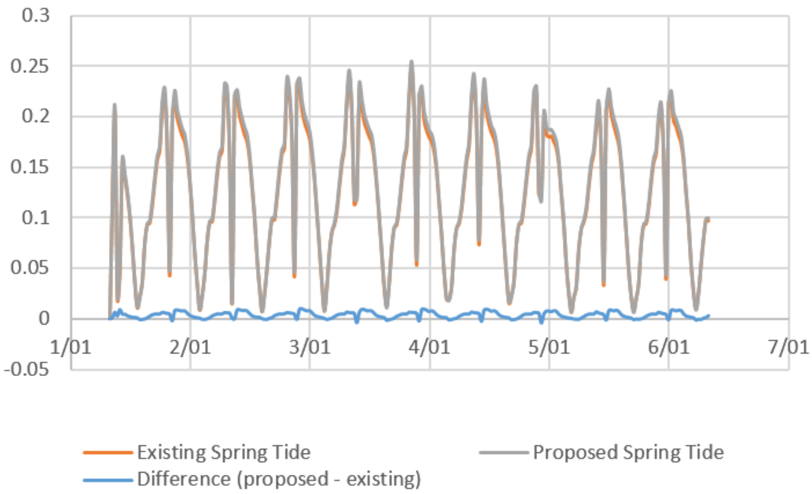
## Spring Tide - Surface Elevation - Data Point 7

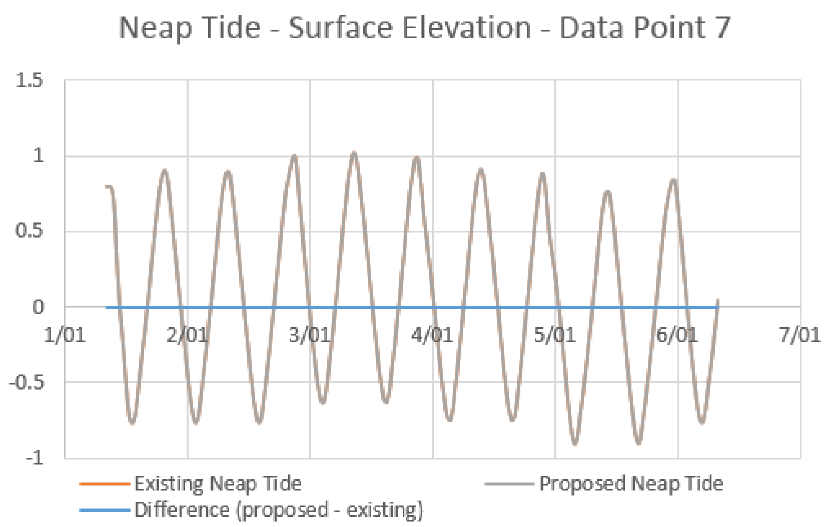


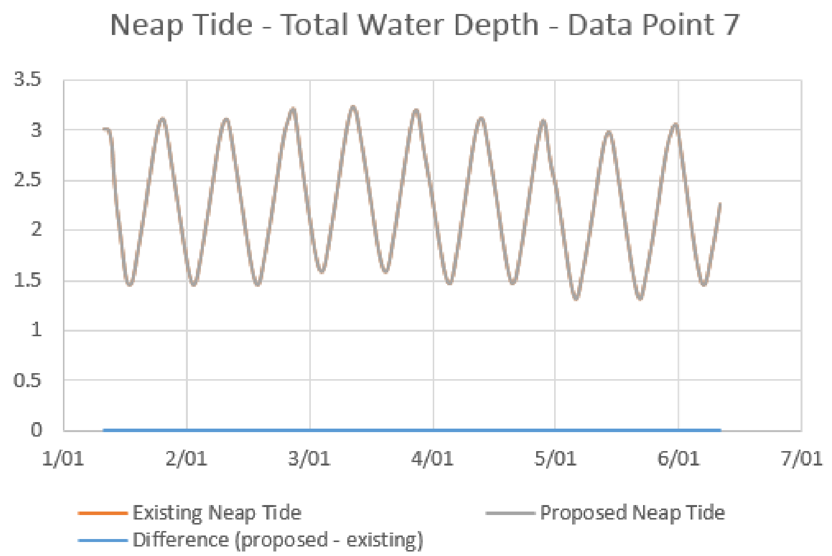

## Spring Tide - Total Water Depth - Data Point 7

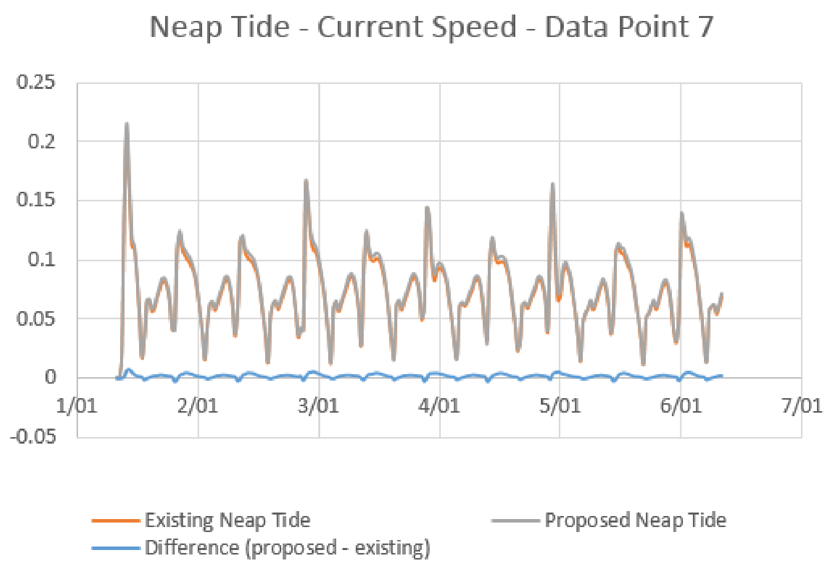


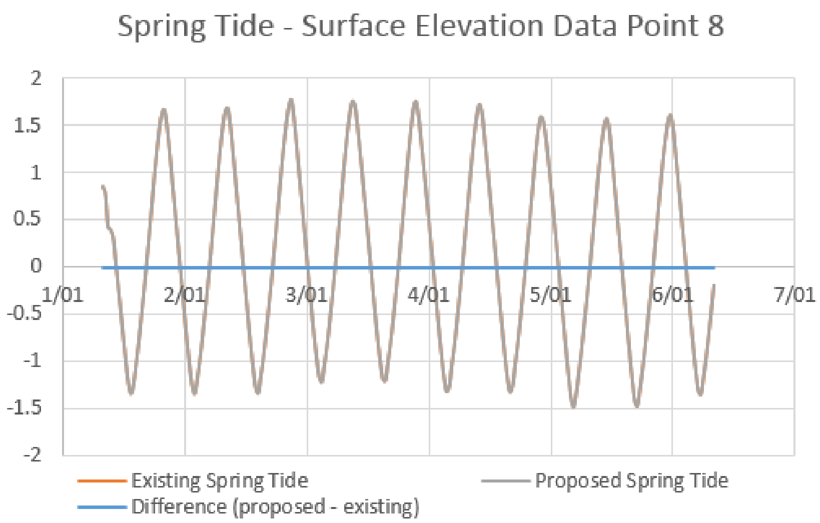

## Spring Tide - Current Speed - Data Point 7



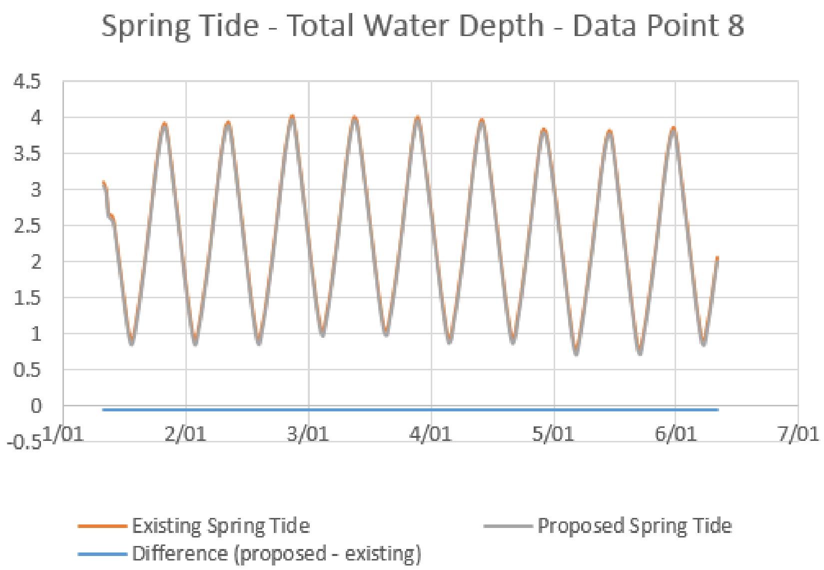

Neap Tide - Surface Elevation - Data Point 7
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Total Water Depth - Data Point 7
3.5
3
2.5
2
1.5
1
0.5
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

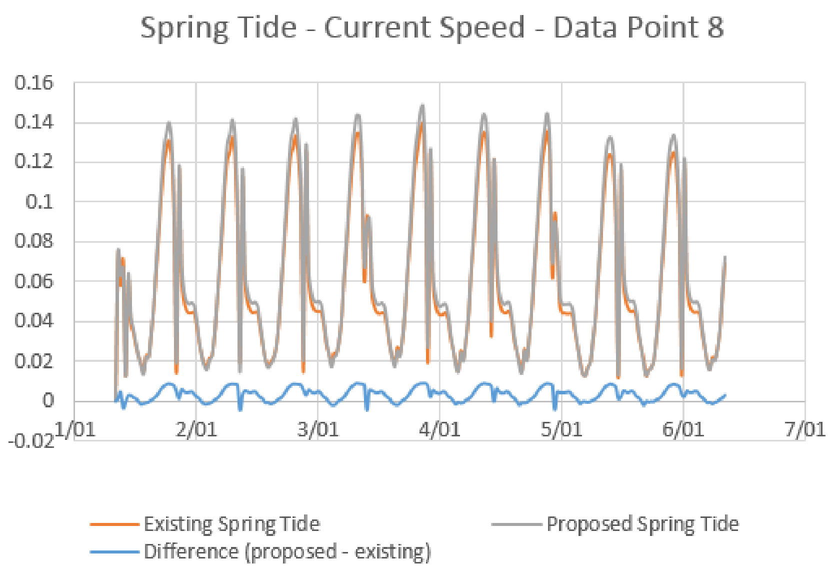

Neap Tide - Current Speed - Data Point 7
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)



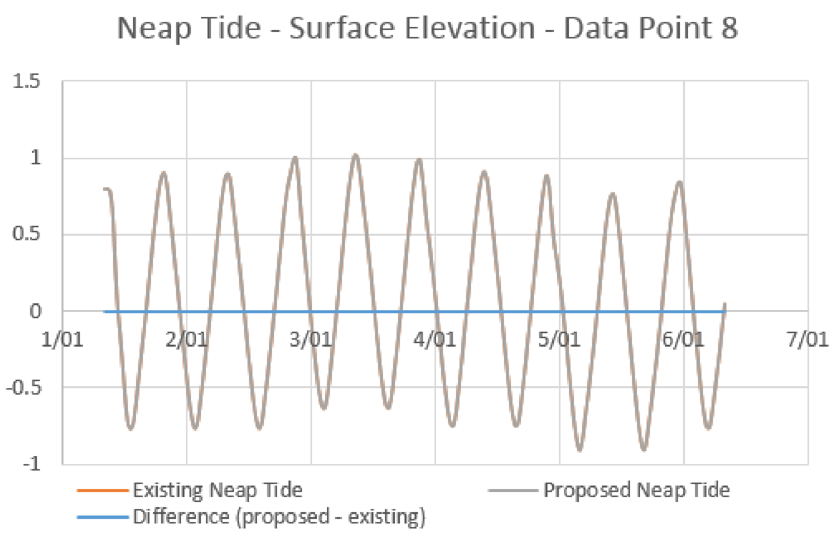
Spring Tide - Surface Elevation Data Point 8
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

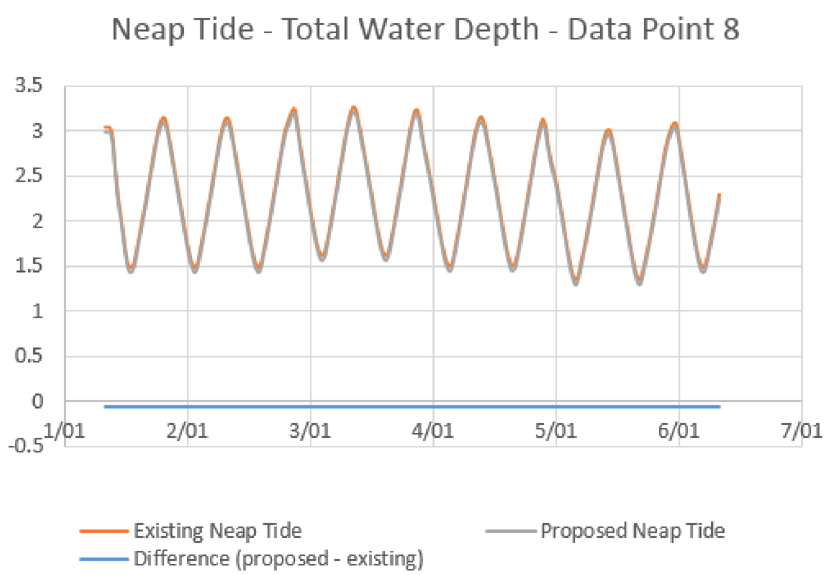



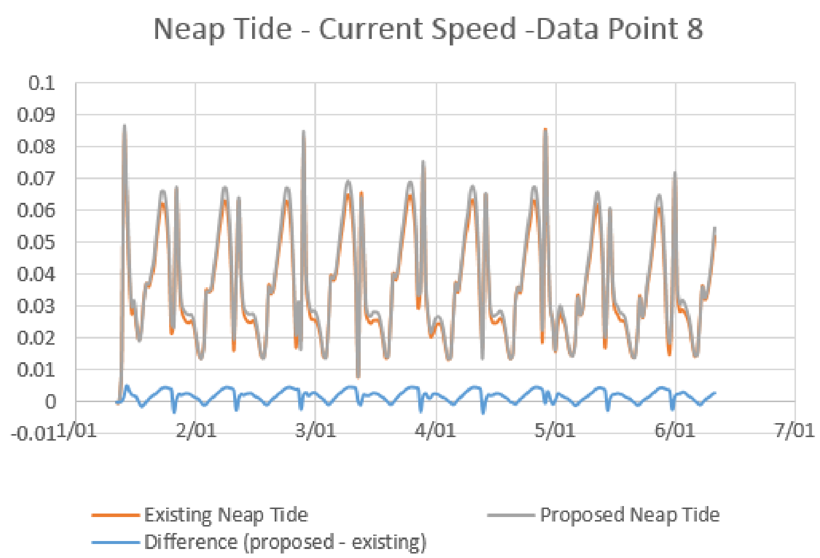
Spring Tide - Total Water Depth - Data Point 8
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

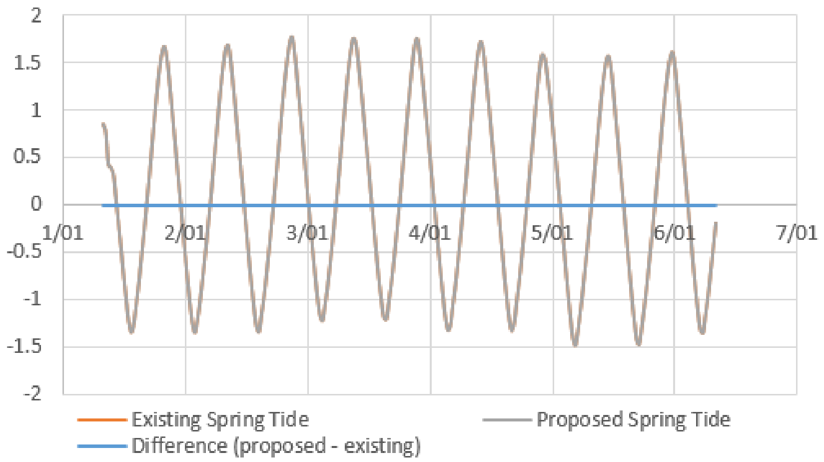
Spring Tide - Current Speed - Data Point 8
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
-0.02
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)


Neap Tide - Surface Elevation - Data Point 8
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

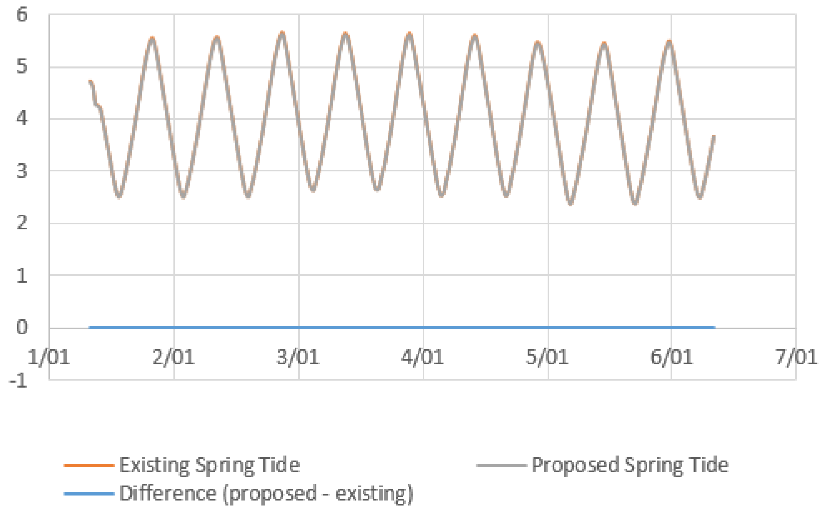

Neap Tide - Total Water Depth - Data Point 8
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Current Speed -Data Point 8
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
-0.01
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

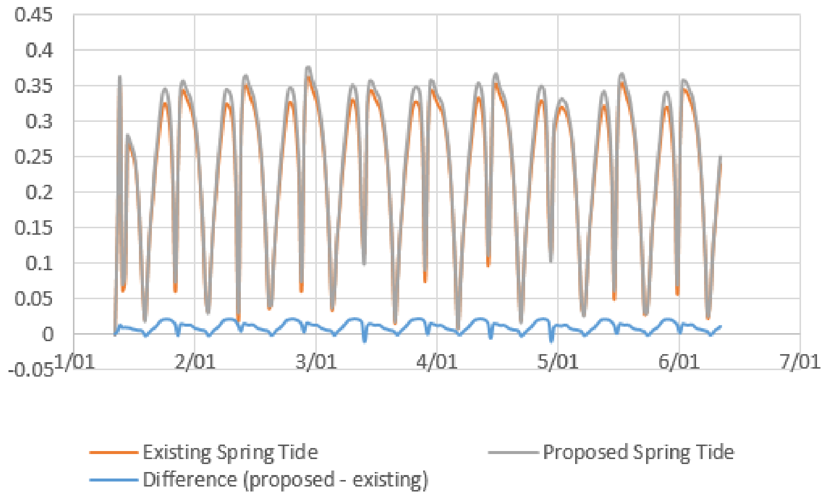
## Spring Tide - Surface Elevation - Data Point 9

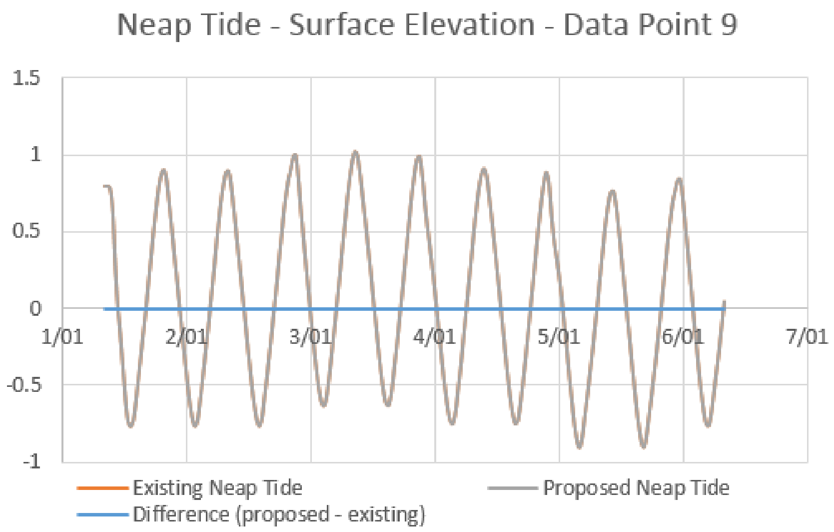


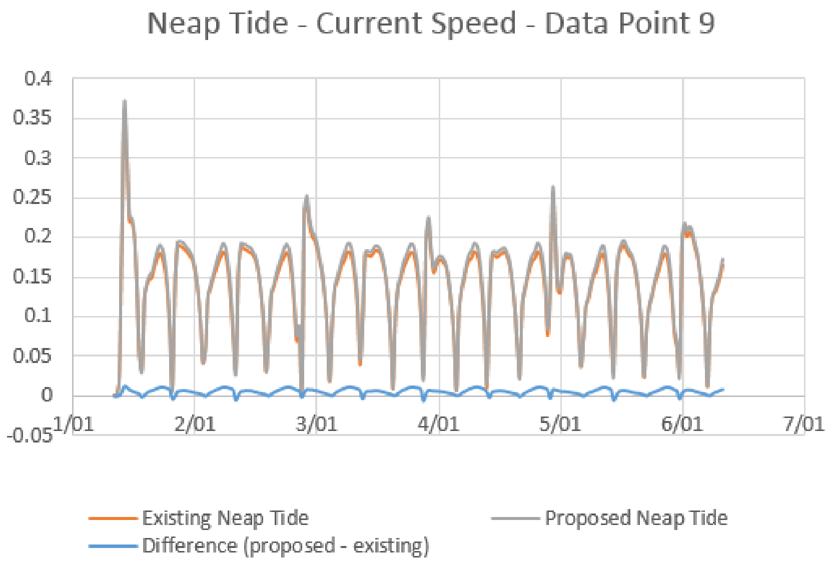

## Spring Tide - Total Water Depth - Data Point 9

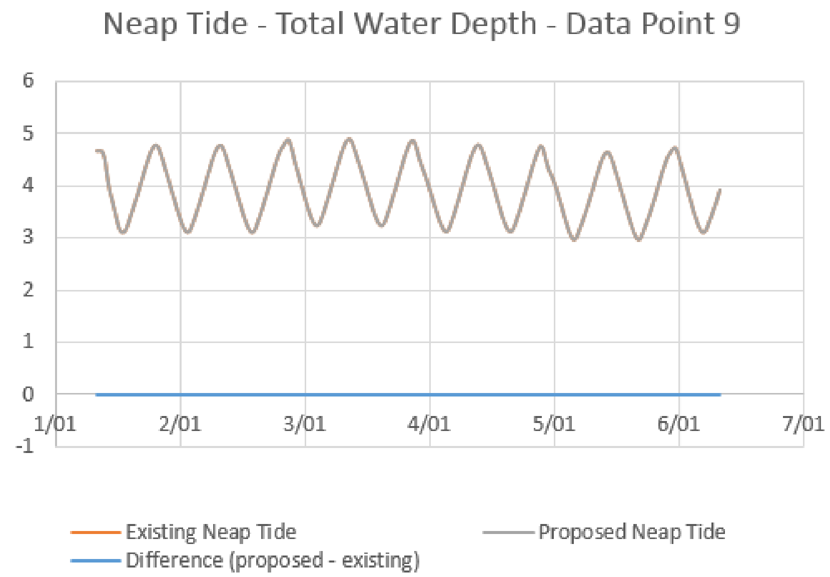


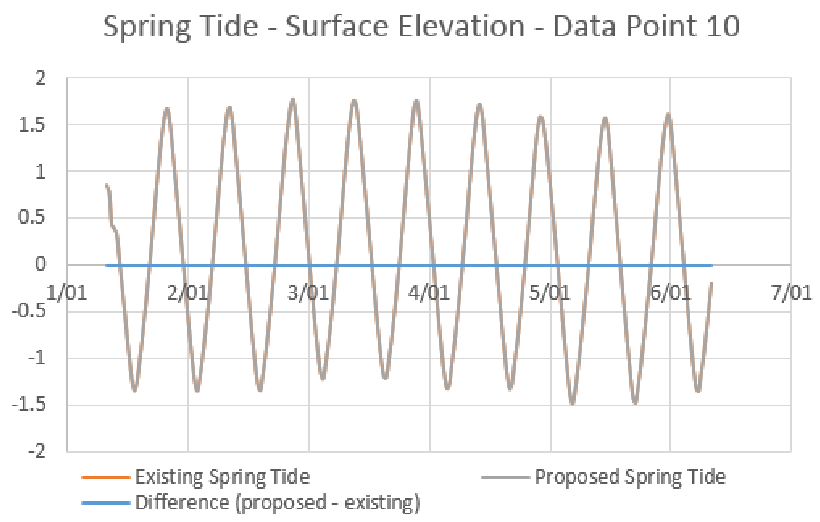

## Spring Tide - Current Speed - Data Point 9

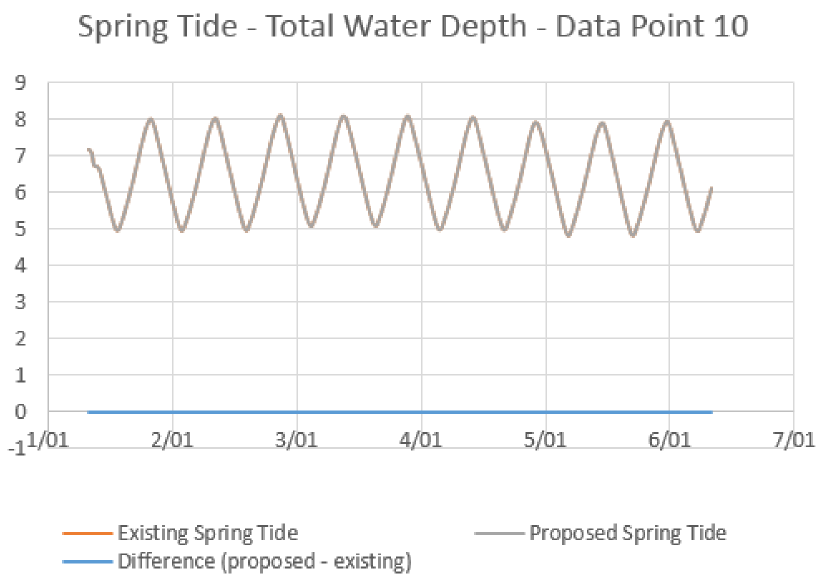


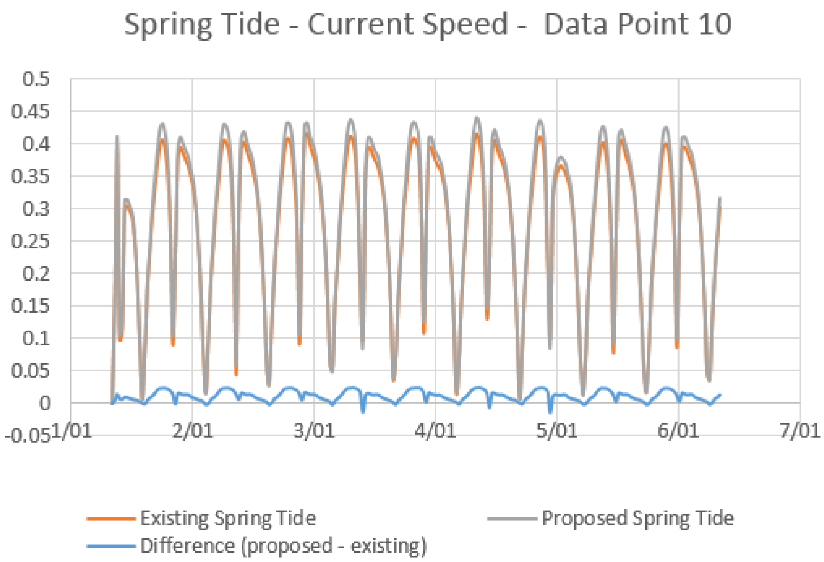

Neap Tide - Surface Elevation - Data Point 9
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Neap Tide - Current Speed - Data Point 9
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

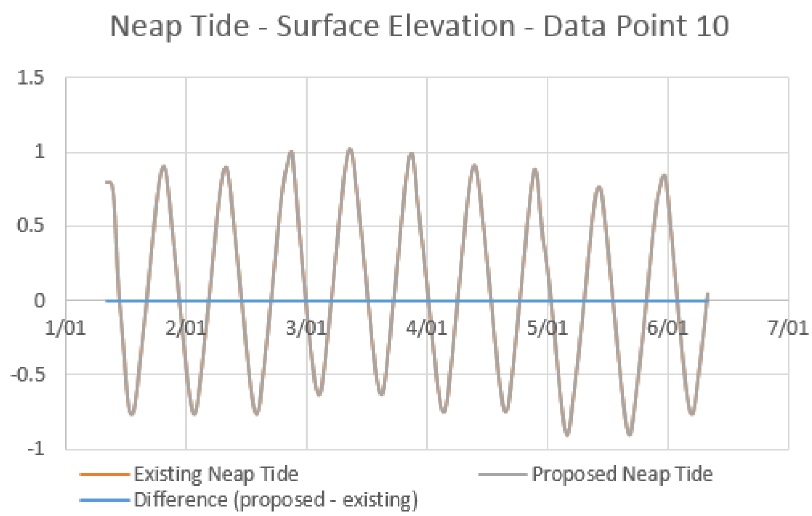

Neap Tide - Total Water Depth - Data Point 9
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)


Spring Tide - Surface Elevation - Data Point 10
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

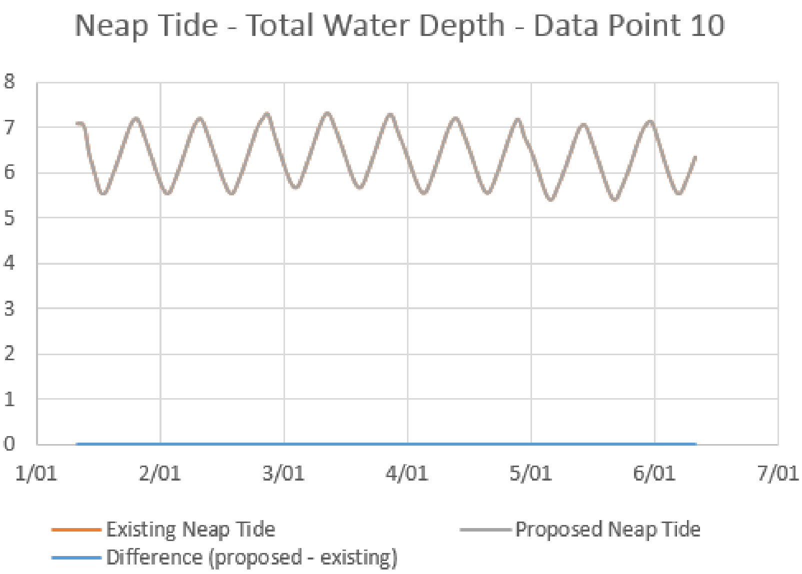

Spring Tide - Total Water Depth - Data Point 10
9
8
7
6
5
4
3
2
1
0
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)



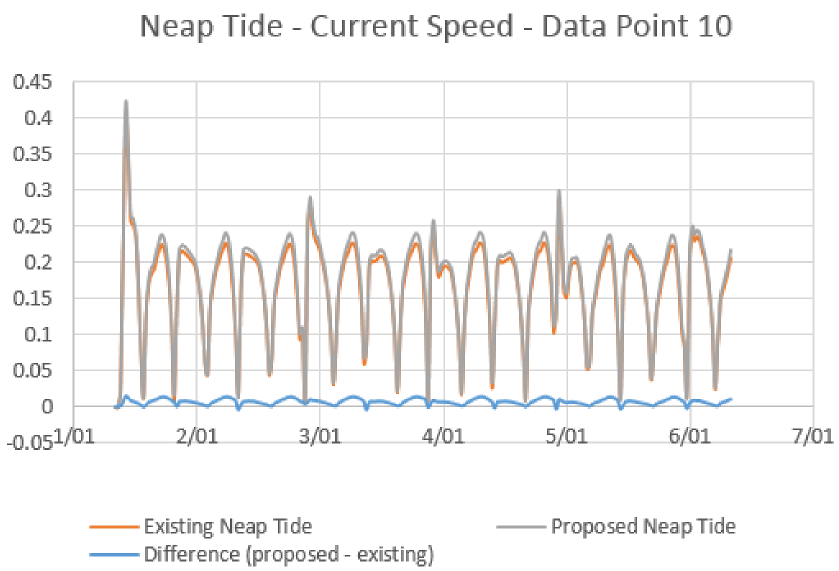
Spring Tide - Current Speed - Data Point 10
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

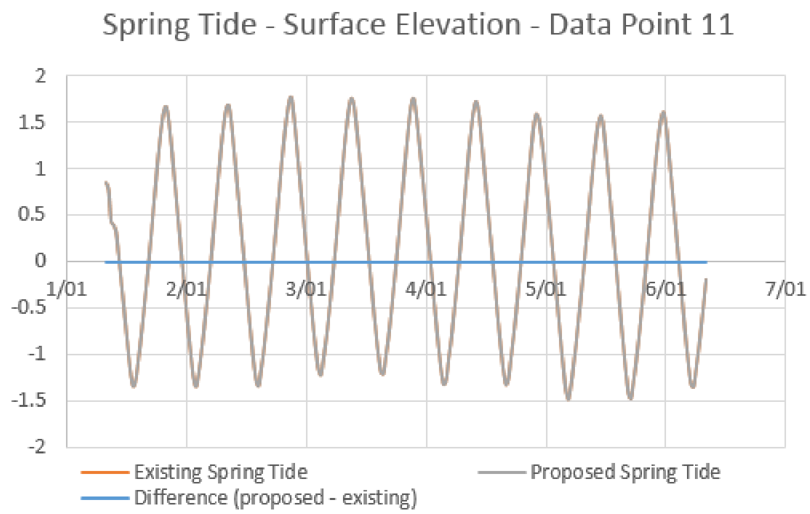
Neap Tide - Surface Elevation - Data Point 10
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

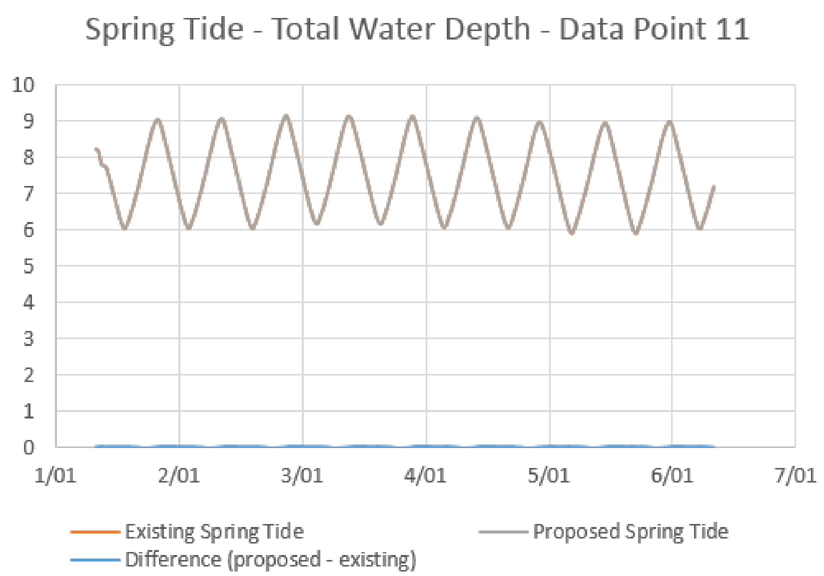
Neap Tide - Total Water Depth - Data Point 10
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)



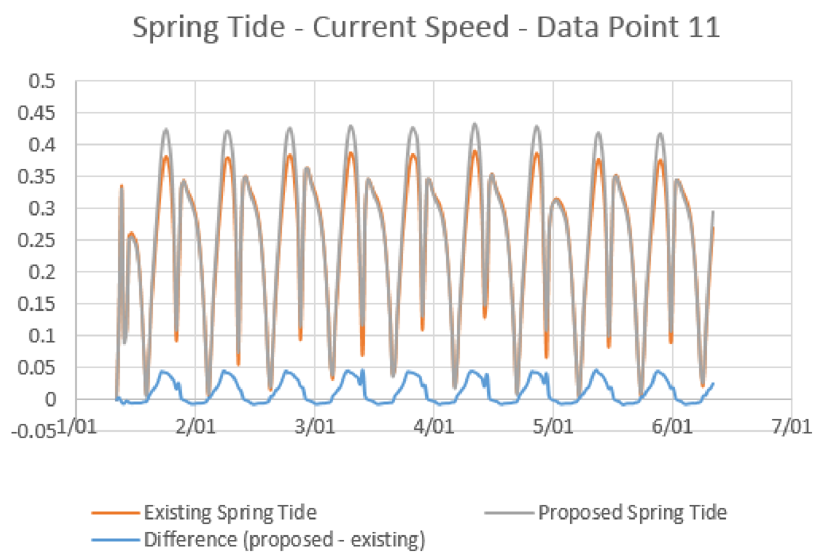
Neap Tide - Current Speed - Data Point 10
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

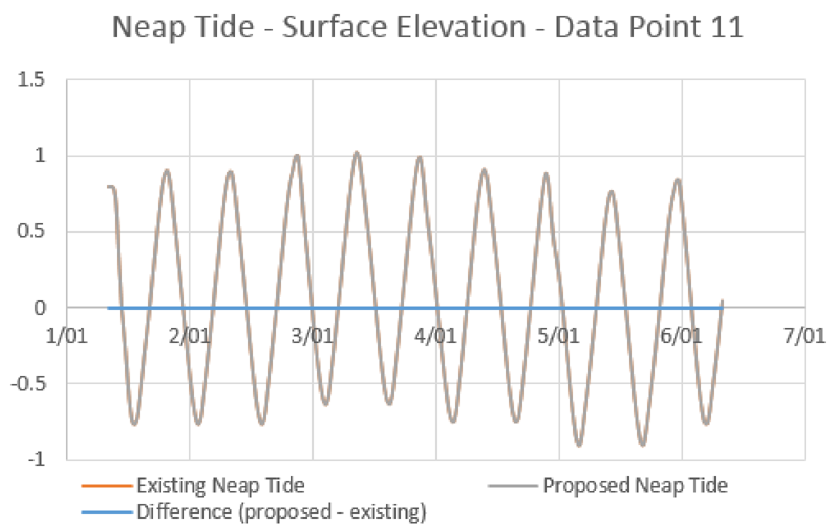
Spring Tide - Surface Elevation - Data Point 11
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

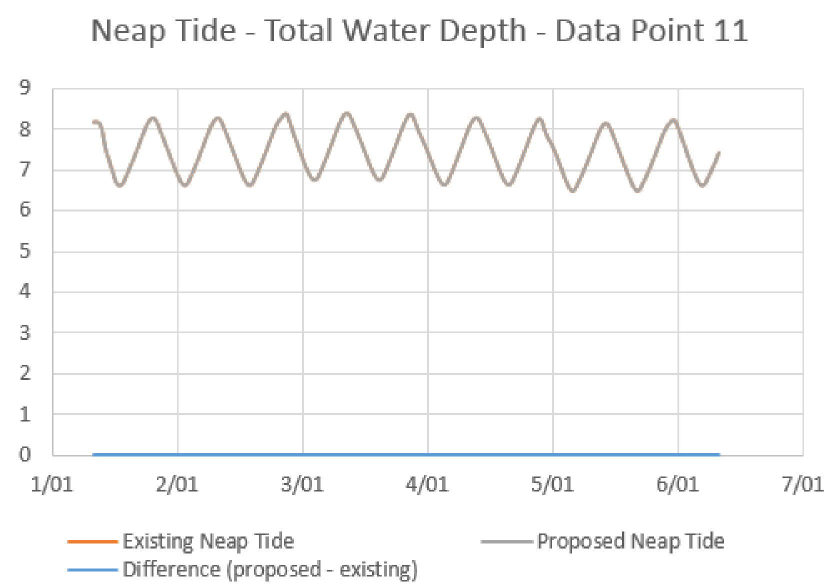
Spring Tide - Total Water Depth - Data Point 11
10
9
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)



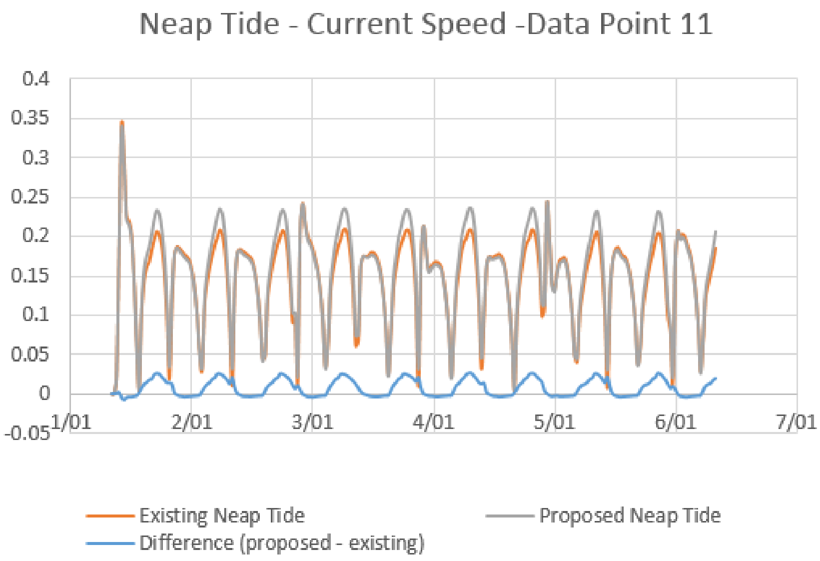
Spring Tide - Current Speed - Data Point 11
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Spring Tide
Proposed Spring Tide
Difference (proposed - existing)

Neap Tide - Surface Elevation - Data Point 11
1.5
1
0.5
0
-0.5
-1
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

Neap Tide - Total Water Depth - Data Point 11
9
8
7
6
5
4
3
2
1
0
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)



Neap Tide - Current Speed -Data Point 11
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.05
1/01
2/01
3/01
4/01
5/01
6/01
7/01
Existing Neap Tide
Proposed Neap Tide
Difference (proposed - existing)

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## Appendix C: Historic charts and oblique photographs

- **Figure C 1 1854 Bathymetric chart (Source: Governor Greys Collection Auckland Library)**
- **Figure C 2 White Aviation Photograph 1949 03 17**
- **Figure C 3 White Aviation Photograph 1955 11 07**
- **Figure C 4 White Aviation Photograph 1955 11 17**
- **Figure C 5 White Aviation Photograph 1963 06 06**
- **Figure C 6 White Aviation Photograph 1973 07 04**


*Figure C 1 1854 Bathymetric chart (Source: Governor Greys Collection Auckland Library)*

*Figure C 2 White Aviation Photograph 1949 03 17*

*Figure C 3 White Aviation Photograph 1955 11 07*

*Figure C 4 White Aviation Photograph 1955 11 17*

*Figure C 5 White Aviation Photograph 1963 06 06*

*Figure C 6 White Aviation Photograph 1973 07 04*

## Appendix D: Historic aerials and satellite images

- **Figure D 1 Aerial from 1959 (Source: Auckland Council GIS)**
- **Figure D 2 Aerial from 1966 (Source: Auckland Council GIS)**
- **Figure D 3 Satellite image, 30 August 2004 (Source: Google Earth)**
- **Figure D 4 Aerial from 2006 (Source: Auckland Council GIS)**
- **Figure D 5 Satellite image, Jul 2006 to Mar 2007 (Source: Google Earth)**
- **Figure D 6 Aerial from 2008 (Source: Auckland Council GIS)**
- **Figure D 7 Satellite image, 28 March 2009 (Source: Google Earth)**
- **Figure D 8 Satellite image, 30 August 2013 (Source: Google Earth)**

*Figure D 1 Aerial from 1959 (Source: Auckland Council GIS)*

*Figure D 2 Aerial from 1996 (Source: Auckland Council GIS)*


*Figure D 3 Satellite image, 30 August 2004 (Source: Google Earth)*

*Figure D 4 Aerial from 2006 (Source: Auckland Council GIS)*

*Figure D 5 Satellite image, Jul 2006 to Mar 2007 (Source: Google Earth)*

*Figure D 6 Aerial from 2008 (Source: Auckland Council GIS)*

*Figure D 7 Satellite image, 28 March 2009 (Source: Google Earth)*


*Figure D 8 Satellite image, 30 August 2013 (Source: Google Earth)* www.tonkin.co.nz

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